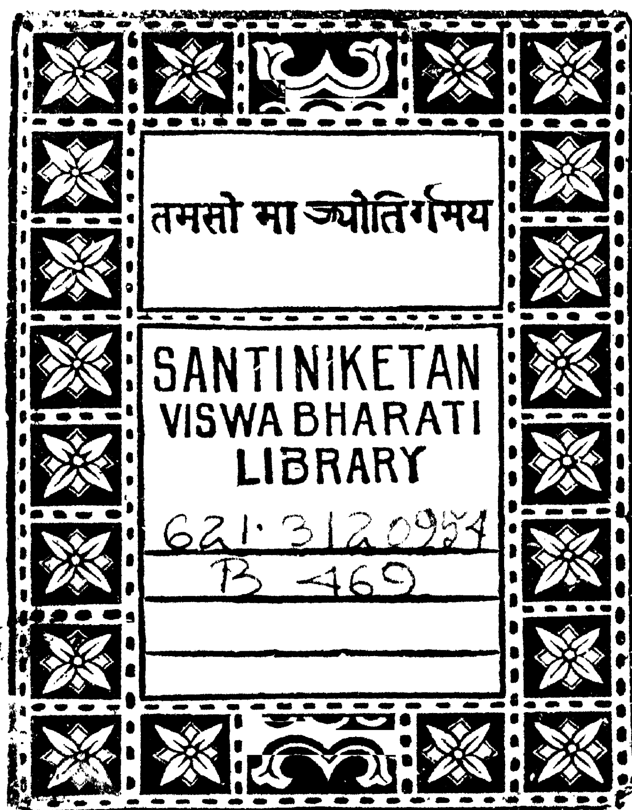

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THE HYDRO-ELECTRIC PRACTICE IN INDIA.

VOL. I

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THE HYDRO-ELECTRIC PRACTICE IN INDIA

BY

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IN TWO VOLUMES.

VOL. I

Published by:

SHIVA NARAYAN CHATTERJEE, B. Sc.

1, LAKSHA ROAD, BENARES CITY.

1936.

PRINTED BY A. BOSE
AT THE INDIAN PRESS, LIMITED,
BENARES-BRANCH.

त्वदीयं वस्तु गोविन्द तुभ्यमेव समर्पये ।
गृहाण सुमुखो भूत्वा प्रसीद पुरुषोत्तम ॥

श्रीकृष्णार्पणमस्तु

मूकं करोति वाचालं पंगुं लंघयते गिरिम् ।
यत्कृपा तमहं वन्दे परमानन्दमाधवम् ॥

রে রাম! তোর জন্যই এই বই। আমার লেখা শেষ না হইতেই তুই আজ বিশ্বনাথের কৃপায় সর্ব্বব্যাপী বিভু। মর জগতে তোকে "হাঁরে রাম" বলিয়া ডাকিলে তুই আদরে "কি বাবা" বলতিস্।

আজ তোর বাণী—

"পত্রং পুষ্পং ফলং তোয়ং যো মে ভক্ত্যা প্রযচ্ছতি।
তদহং ভক্ত্যুপহৃতং অশ্নামি প্রযতাত্মনঃ॥"

শুনিয়া হৃদয়ের নিবিড় বেদনা ও ভালবাসার সহিত তোর প্রসাদ তোকে দিলাম।

PREFACE.

The Hydro-Electric Practice in India was originally intended for the B. Sc. students in engineering of the Benares Hindu University and to guide the engineers entrusted with the task of designing and constructing hydro-electric developments or reporting on their commercial success. The following pages comprise, in a systematic manner, those principles of hydraulics and electrical engineering underlying the design of hydro-electric schemes. The principles are illustrated throughout by reference to current practice in the most important hydro-electric schemes of India. An earnest effort has been made in this treatise to enable them to successfully work out such schemes from start to finish and solve all problems that may face them in the design, construction and management in the course of their daily practice.

Details concerning the principles underlying the design, operation and management of electrical plant will be found fully dealt with in my works on Applied Electricity.

Every effort has been made to illustrate the text throughout, by photographs and drawings, in order that readers may form as clear an idea as possible of the places described and subjects dealt with.

In a work of this kind I have necessarily drawn freely from all sources of information, and I believe that due acknowledgment has been made. However, in some instances, search for the original has proved fruitless, and I apologise to all authors and publishers who may find their work used without any definite reference, such omissions being unintentional.

I acknowledge with thanks, however, the great benefit I have derived from the works of the following authors, among others, and I take this opportunity to thank their publishers:

(1) Von Schon : Hydro-Electric Practice.
(2) Rushmore and Lof : Hydro-Electric Engineering.
(3) J. W. Meares : Hydro-Electric Development.
(4) W. T. Taylor : Water-Power Engineering.
(5) Creager and Justin : Hydro-Electric Handbook.
(6) A. H. Gibson : Hydro-Electric Engineering.
(7) Lahmar Lyndon : Hydro-Electric Power.
(8) C. T. Barlow and J. W. Meares : Hydro-Electric Survey of India Vol. I.
(9) J. W. Mears: „ „ „ Vols II, III.
(10) Daniel W. Mead : Water-Power Engineering.
(11) Richard and Muller : Hydro-Electric Engineering.
(12) F. T. Morse : Power Plant Engineering and Design.

To the publishers of the following journals and to the manufacturers special thanks are due for many illustrations and permission to use their publications.

(1) The Journal of the Institution of Electrical Engineers (London), contributor : Mr. Alfred Dickinson, —(the Tata Hydro-Electric Power Supply Company).
(2) The Journal of the Institution of Engineers, India, J. F. Heath, (The Andhra Valley Scheme.)
(3) The Civil Engineering (London), (Mettur Project.)
(4) The Engineering—London : Water Turbines for Shannon Generating Station (Mandi Scheme, and Shiva Samudram.)

(5) The Engineer – London : Water Turbines for the Shannon Power-house.
(6) Messrs. Metropolitan Vickers, Brown Bovery, Boving & Co., and the General Electric Co. and others for the use of their instruction books and plates.

I further acknowledge with thanks the kind permission, sympathetic co-operation, and timely response of the following experts for helping me with all necessary information in connection with their schemes.

(1) Sir William Stampe – The Ganges Hydro-Electric Scheme.
(2) Colonel A. Bell – The Mussoorie Hydro-Electric Scheme.
(3) The General Manager – The Shivasamudram Scheme.
(4) The General Manager – The Tata Schemes.
(5) Mr. L. C. Bose, Chief Electrical Engineer, Kashmir.
(6) Lt. General Krishna Shumsher Jung Bahadur Rana.
(7) Colonel Chet Shumsher Jung Bahadur Rana.
(8) Mr. R. G. Kilburn, Chief Electrical Engineer, Nepal.
(9) Major Hem Shumsher Thapa.
(10) Mr J. N. Ganguly, Superintendent, Sundarijal Power-house, Sundarijal, Nepal.

(6)–(10): For the Nepal Schemes.

I may state that besides drawings and descriptions, financial reports, which are very difficult to procure, have been available to me only through their kindness.

To the Bombay Electric Supply and Tramway Company, I am indebted for much information regarding Sub-Station Practice in India.

The last stages of revision of the book were undertaken under the stresses of a great domestic bereavement, but I trust no serious flaws have passed undetected. If the book contain errors, I shall be grateful to any reader who will point them out to me.

Engineering College,
BENARES,
January 1, 1936.

B. C. CHATTERJEE.

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THE HYDRO-ELECTRIC PRACTICE IN INDIA

CHAPTER I

GENERAL INTRODUCTION

The prosperity of a country is measured by the useful work done by utilizing energy stored in various forms in Nature, and cheap power is the basis of national prosperity and one of the bases of economical manufacturing and of high social level. Nature receives its store of energy from the Sun, Who is thus the ultimate source of all we have—body, energy, mind, health, wealth and existence. Not a moment can we dream without Him. Energy in whatever form—waterfall, oil, and coal—is the direct gift of the Sun. So long as the Sun continues to shine, the cycle of evaporation, condensation, and fall to sea-level again will continue to provide rain and power, and food and energy, long after the natural fertilizers are used up and the energy in coal and oil has been exhausted. No wonder we find :—

आदित्याज्जायते वृष्टिः वृष्टेरन्नं ततः प्रजाः ।—मनु ।

वृष्टिप्रमोचनविनिग्रहहेतुभूतं[1]

त्रैलोक्यपालनपरं त्रिगुणात्मकञ्च[2] ।

प्रातर्भजामि सवितारमनन्तशक्तिं

पापौघशत्रुभय[3]रोगहरं[4] परञ्च ॥

Ages ago men worshipped the Sun, and called the first day of the week after Him. This practice still continues. They had in them the instinct of worship, as they took the Sun to be the visible expression of God.

Thus the celebration of Yuletide in Europe, and the Sun or Mitra-worship by the Parsi Community, and our Ithu Puja and Mitra-worship[5] of Bengal, etc., are in grateful recognition of the infinite blessings bestowed by the Sun—the great giver of health, wealth and energy.

Free gift is seldom appreciated. Food and energy go crying from our doors. The opportunities pass off unnoticed. Although

1. Cause of rain.
2. Source of Potential, Kinetic and Sentient energy.
3. Destroyer of darkness, ignorance and other enemies of men, including microbes.
4. Helio-therapy and Sunbath.
5. मित्रसप्तमी ।

we have not many perennial streams all over the country, we can store up the rain-water, and develop waterfalls, as has been practically demonstrated by the great Tata Company. The *rain-water*, if properly utilized by making suitable bunds,[1] will help agriculture, and will increase the food-supply, ensure the growth of materials required for industrial purpose, provide the fertilizers to the agriculturists. The *energy of the water* will supply cheap power, and thus help the development of various industries, give comfort and joy to the pedestrians and householders by supplying power for light and fans, for heating and cooking, often at a cost unapproachable by steam or any other power. The energy which we may get from the rain-water, if properly utilized, would give us all comfort that may be due to cheap power, would make our country rich, and impart life to the despondent millions, while the improvement in agriculture will drive famine out of India. We should be thus sinners for neglecting God's free gift—the rain-water.

Development of industry, comfort in life, improvement in various arts and manufactures, in fact, the most important factors in advancing the course of civilization in any country, depend upon cheap power, which, in its turn, so much depends upon the utilization of the gifts of God—the waterfall.

Let us briefly review how far Ancient India was advanced in Hydraulic Engineering. *

We may briefly notice the hydraulic works of the great sages and kings of Ancient India; after four generations of strenuous effort of the kings of Ajodhya, our great Bhagirath brought down the Ganges from the Himalayas, and directed the course of this great river to the Bay of Bengal.

तच्च गङ्गावतरणं त्वया कृतमरिन्दम ।
अनेन च भवान् प्राप्तो धर्म्मस्यायतनं महत् ॥

१३ । ३४ आदि, रामायणम् ।

Thus our national poet sings :—

कत नग नगरी तीर्थे हइल तव चुम्बि चरणयुग मायि ।
कत नर नारी धन्य हइल मा तव सलिले अवगाहि ॥

The Ganges is given a twist in front of our great city of Baranasi, perhaps the first planned town in the world, so that it flows towards the north. Two small branches, Baruna and Asi, were tapped from the Ganges, at the two extremities of the city, to carry away the excess water at the time of flood to distant

1. (आरुणि) "गच्छ केदारखण्डं बधानेति" । २२ । ३ । आदि, महाभारत ।

* *Vide*—"Hydraulic Engineering in Ancient India," by the Author.

places. Silt, the potent fertilizer, is thus carried far to fertilize a plot of land which became so productive as to feed the entire population within the embrace of the *rivulets*—Baranasi. Thus goes the proverb 'none dies of starvation within the precincts of the dominion of Mother Annapurna.' Not only this, but there were hundreds of well-constructed tanks of considerable capacity, all in connection with the Ganges, to hold the flood water, when occasion demanded, and save the city of Vishweshwara from ruin.

Many illustrious names may be mentioned, whose works were not only gigantic in themselves, such as none but a superman could conceive, but were productive of the most important results to the inhabitants of our country.

Parasuram, the great sage, paved the way of the Brahmaputra.

Thus we find :—

ब्रह्मकुण्डात् सृतः सोऽथ कासारे लोहिताह्वयः ।
कैलासोपत्यकायान्तु न्यपतद् ब्रह्मणः सुतः ॥
तस्यापि सरसस्तीरे समुत्थाय महाबलः ।
कुठारेण दिशं पूर्व्वामानयद् ब्रह्मणः सुतम् ॥
ततो परत्रापि गिरिं हेमशृंगं विभिद्य च ।
कामरूपान्तरं पीठमवाहयदमुं हरिः ॥—कालिकापुराण, ८४ अध्याय ।

So were the Krishna, Kaveri, Godavari, Mahanadi, and Kausaki, trained or directed by the great souls of India in the past.

The 18 *nalas* in Puri and many other canals, which ultimately developed into rivers and rivulets, were the works of Rajas and sages, who constructed them to prevent flood, and utilize the water for irrigation in times of drought. The great virtue of making tanks, wells, etc., for the inhabitants of the place, has been religiously sanctioned, and carried into effect, and is being done even to-day.

चतुर्व्वर्गमहोदानजन्यफलसमफलप्राप्तिकामः । (श्रीविष्णुप्रीतिकामो वा)
इमं जलपूर्णपुष्करिणीजलाशयं वरुणदैवतं सर्व्वभूतेभ्योऽहमुत्सृजे ।
ॐ देवपितृमनुष्याः प्रीयन्ताम् ।
सर्व्वभूतेभ्य उत्सृष्टं मयैतज्जलमूर्ज्जितम् ।
रमन्तु सर्वभूतानि स्नानपानावगाहनैः ॥
सामान्यं सर्वभूतेभ्यो मया दत्तमिदं जलम् ।
रमन्तु सर्व्वभूतानि स्नानपानावगाहनैः ॥

कूपजलाशयप्रतिष्ठा, etc., is also similarly recommended.

Similarly, numerous tanks, lakes, aqueducts, and small tunnels are found all over India to contain or conduct water. The large irrigation tank in the foreground of the Dagaba is probably

the oldest one in India, as it was constructed before the time of Asoka. During the Buddhist period some Great Officers of State superintended the rivers, measuring the land, as was done in Egypt, and inspected the sluices by which water was let out from the main canals into their branches, so that everyone might have an equal supply of it. The Greeks bear testimony to the existence of canals and reservoirs which were maintained and repaired by the kings and supervised by royal officials. They built embankments and reservoirs of water for the good of the agriculturists. In some cases the Government helped co-operative irrigational works. A late inscription of the time of Rudradamana bears testimony to the creation of the lake Sudarsana under the direction of Chandragupta's Yavana viceroy, Pusyagupta. The reference here is to the Girnara lake, which was also *reconstructed* in the days of the Guptas. Mahipal Dighi was constructed by a king of the Pal Dynasty. Ram Sagar in Dinajpur, Sagardighi of Murshidabad, Bindu Sarobar, and numerous lakes and tanks in Bengal, Dhakouni in Gawalior, are yet in existence to relieve human suffering. In Hydrabad Nizam Sagar, in Mysore Krishna Raja Sagar, show the works of our great Rajas and Maharajas in more recent dates. Udaipur is called a city of lakes, which owe their existence to the Maharanas of Udaipur. In all these flood used to play havoc in the villages bordering the lakes ; small dams were erected, and the water was confined within the lakes and tanks ; and the water, thus stored and utilized in irrigation, saved the inhabitants from ruin. In hilly places crude water-wheels have been worked for small flour mills, etc., from time immemorial. But hydro-electric works are of very recent date. The first big water-power utilized in India was, to run cotton mills, at Gokak in 1886, and the first hydro-electric plant was installed at Darjeeling in 1897. At present, however, we find that not more than 0·7 per cent. of the water-power, given to us with rain, has been utilized. The total per capita distribution of Watts is less than even one in India and 7·6 sq. miles subscribe only one kW.—with her teeming population of 350,353,678 (The Times, London, Monday, Sept. 21, 1931) ; and with abundance of Nature's gifts she is the poorest in the world—she is the weakest ! Her condition is most deplorable. Our serious effort should, therefore, be directed to wake up and utilize God's gifts to the full, and, through our hard work, show response of her life.

Present-day Conditions

Now-a-days there is a great demand for cheap power widely distributed, and hydro-electric plant undoubtedly serves the end in

many localities. In foreign countries water-power has been utilized to run hydro-electric plants from 5 kW. to 1,000,000 h. p.; they utilize every opportunity however small or great for running mills. Thus we find that -(1) Aberayron (Cardiganshire) works only a 5 kW. water turbine and one 30 kW. oil set to supply power and light to 195 subscribers in a place having a population of 13,000. This supplies a unit for lighting for less than annas 4. (2) Aberangell (Montogomeryshire) utilizes a waterfall to run one 14 kW. and one 18 kW. set to supply 110 consumers in a place containing a population of 400. (3) Ontario Scheme—This is a **co-operative municipal ownership** enterprise, province-wide in its field, operating through the agency of independent commission control and administration. It is one of the world's largest electrical undertakings. The Commission generates and transmits power on a wholesale scale to about 550 municipalities, of which 325 are urban municipalities—including 23 cities with a population of 10,000 or over, and 49 towns with a population ranging between 2,000 and 10,000—and 225 are townships. The 23 cities utilize about 80% and the 49 towns about 12% of the total power distributed by the Commission. The remaining smaller towns, townships and villages together, utilize less than 10% of the power distributed by the Commission. It serves about 40,000 sq. miles. The aggregate peak load supplied by the Commission exceeds 1,000,000 h. p. It operates 22 water-power plants with an ultimate capacity of more than 1,000,000 h. p., and, with new plants under construction and additional power to be supplied by contract, has provided for a total power supply of 1,400,000 h. p. Its main transmission lines for distributing wholesale power total more than 3,800 miles, and include more than 600 miles of steel-supported 110,000 v. lines, and 230 miles of 220,000 v. lines.* *Consider the production of the country which utilizes such enormous power.* Large hydro-electric works are much the same in principle as the smaller ones. When power is available at a cheap rate from a public supply, there is no incentive for private owners to put down smaller plants. In places where there is no power or power is very costly, a company will take to power supply, and a factory will get its own power plant; but if, in such places, water-power can be supplied very cheap, everybody will subscribe power from the hydro-electric plant; and the scope for such power developments in India is ample. Any running stream of water is a potential source of energy, but it does not mean that it is worth developing or even capable of development.

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* Adopted from Electrician's Annual Tables of Electricity Undertakings, 1933.

So far as prime mover sets are concerned, the hydro-electric machinery may be less costly than other types of prime movers. But the incidental works and accessories of a hydro-electric power scheme, such as dam, reservoirs, etc., are often very costly ; so that, on the whole, such schemes are likely to be considered uneconomical. But, on mature consideration, the saving of fuel will often turn the scale in favour of the hydro-electric scheme. Steam-power is dependant upon fuel, which raises the steam in the boiler ; whereas, water-power depends upon the product of two factors, head and quantity of water. Very often both quantity and head are, to a great extent, fixed by Nature, but an artificial head may be created, and the floods of rainy seasons may be stored against the drought of summer, if the natural condition is favourable.

Electrical energy is rapidly replacing every other form of energy in all branches of industry, and this fact has been widely realized all over the world. What a tremendous consumption of oil and coal—the natural resources of fuel—is necessary to generate this power ! The immediate and remote drastic effects of this natural fuel consumption on the industrial welfare of the nations, are obvious. Thus, in order to preserve the natural fuel resources of the world, without in any way retarding the progress of national industries, is to utilize the enormous energy of numerous waterfalls, which is being wasted at present. It is estimated, according to a recent pamphlet of the "Mannesmann Company" of Germany, that the total water-power available on the earth is capable of producing 1,100 millions kW. with an annual output of 4 billions kWh.; but, unfortunately, only a small fraction of it is, now-a-days, utilized. It is well-known that the world has made use of 5% of its potentialities of water-power, and India has not even harnessed over 0·7% of its potential water-power, which was estimated to be $2{\cdot}7 \times 10^{7}$ h. p.

As in many other things, Nature has endowed India not only with potentialities of water-power, but also with fuel resources. Hence, in the provinces like Bengal, where the fuel is very cheap, very careful investigation of facts and figures are to be made before a heavy investment on the hydro-electric project is undertaken. The natural fuel resources in Southern India and in the Punjab are very meagre, but Nature has given various waterfalls all over ; hence, from the consideration of economy, these provinces are quite suitable for the hydro-electric project.

At the present time India is striving much for the development of industries ; and the demand for a reliable supply of power at economic values is a sheer necessity. The rapid growth

of power demand has raised the cost of fuel, and this, in its turn, has increased the activity in the development of all resources of water-power in different parts of India, wherever water-power is, at all, available, and cost of coal high.

It is a common belief that any water-power developed will be profitable. As an undeveloped water-power is a continual waste of energy, it is commonly assumed that the saving of this waste is bound to result in a profit to those who acquire the property and develop the power. But this is not the fact although *bund* will help agriculture.

Hazards of Water-Power Investment

It should be carefully borne in mind that development of water-power is not a simple method of assuredly capitalizing the waste energy of streams. The hazards involved both in the construction of such properties and in the contingencies of operation and maintenance, are considerable. With the advent of electrical transmission, coupled with the popular conception that water-powers were excessively profitable, and that, by means of such developments, the waste energy of water could be advantageously turned into dividends, investers eagerly seek water-power investments. The results have not always been satisfactory. It is not enough that the power be constant and sufficient in quantity, that the plant is well-designed, and that the cost of the same is reasonable. There must be a **market** in which the power can be utilized to advantage, and the price at which it can be sold in competition with all other sources of power, must be sufficient to pay fixed charges, and all other expenses involved in the construction and operation of the plant, and afford a fair return on the investment to those who assume the risk of the undertaking. *The entire commercial success depends on its financial feasibility.*

—— ——

CHAPTER II

FEASIBILITY OF THE HYDRO-ELECTRIC SCHEME.

Summary

First understand and examine what is a hydro-electric scheme, and then study if it is feasible and practicable. To utilize water-power, first scrutinize the conditions under which water-power is available. Consider the basis of water-power which depends upon the definiteness of flow and the nature of the fall. The cost of the scheme depends upon the nature of the development, which is divided into three types as high, medium, and low. The types should be carefully understood. See that the schemes are worth consideration only under certain conditions, and hence the salient points and limitations should be carefully balanced. Before reconnaissance and costly survey is undertaken, a number of important points must be further examined ; and if the scheme appears to be workable, consider the leading factors governing the choice of the project, and then proceed with the reconnaissance, survey of the site, and finally consider the general report on site. Study the causes of failure. Next consider the feasibility and practicability of the scheme ; and when everything is favourable, see if it can compete with a steam-power plant.

For this, determine the consumption of fuel in a steam plant of the same capacity, the cost of coal in the locality, and finally the cost per unit of energy supplied, and compare the cost of a unit with that in a hydro-electric scheme ; also the cost of the installation per kW., including the cost of development and transmission.

If it finally compares favourably with the steam plant, there are four more factors to be considered—(1) freight of materials and machinery to site, (2) cost of transmission of power, (3) load factor, and (4) diversity factor. If finally the scheme is feasible and practicable, proceed with the development.

Uses of Hydro-Electric Power

First let us see what is the use of hydro-electric power. We find that the chief uses of hydro-electric power in India are mainly in electro-metallurgy, electro-chemistry, agriculture, lighting, heating, fan, traction and other industrial applications. Wherever power from a hydro-electric source can be

developed cheap enough, owners getting their small power from uneconomical fuel plants may be induced to shut down their plants and get power from the hydro-electric company. A number of industries may further be started in the immediate *neighbourhood* of the power distribution area. A hydro-electric plant may thus very often create its own market. The *electrification of railroads,* having proved a success in the initial experimental stage, is bound to grow with the cheap supply of electric power. India being mostly an agricultural country, the application of cheap hydro-electric power in many forms may be popularized in *agriculture.* **Irrigation** with cheap power will materially alter the quantity and quality of the field produce. In those parts of the country where rainfall is scanty, untimely, and uncertain, the *application of electric motor*, with the resulting advantages of running smoothly, its flexibility and reliability, in pumping can do much to *diminish the ravages of famine,* and even where the holdings are small, the agriculture can be made more intensive, if not extensive, raising a larger number of crops per annum than would be otherwise possible. Many *electro-chemical* and *electro-metallurgical industries* would be opened and materials of every-day use would be available with the aid of cheap electrical power —for instance, aluminium, chromium, calcium carbide, magnesium, sodium, etc. The *electric smelting* of indigenous iron ores and the production of steel and its alloys, electrolytic copper and zinc, and the refining of other metallic ores are among some of the mining industries likely to be affected. An increasingly large amount of electric power is being used, as in Norway and Canada, for the *production of nitric acid and commercial nitrates* directly from *atmospheric nitrogen* to serve as *artificial manures* of great value, which increase and maintain the fertility of arable land. The crying need for the *industrial development* of India in industries like cotton, jute, sugar, and oil mills, woodworking and paper-pulp, etc., would have a bright future if power could be cheaply supplied from the numerous potential water-power resources available. In some quarters, as in Jeypore in the Vizag District, enormous water-power is available from waterfalls, but the authorities apprehend that they do not find sufficient industry to utilize the power. We may point out to them the *Ontario Scheme,* very briefly stated in the General Introduction, Chapter I, and suggest to them some of the above industries, including agriculture in which they can well utilize their power however vast it may be. Now that we have seen some of the uses of water-power, we proceed to examine the scheme in detail.

Conditions under which Water-Power is Available

(1) The mere existence of rivers, reservoirs, and waterfalls, capable of delivering vast quantities of energy, is not sufficient for a profitable hydro-electric undertaking, if the nature of the country containing them be not favourable for the purpose. For instance, the Marble Rock Fall, near Jubbulpore, is roughly a fall of about 35 to 40 ft. The whole water of the Narmada River falls down that depth with rising foam, for which it derives the name of Dhua Dhar, where "smoky outburst" in the vicinity at the fall gives the name of the fall, but it has not been utilized yet for want of industries to absorb the power.

(2) The slope of the stream may be very small, say, a few inches to the mile, as is the case with many large rivers in flat countries. Here, small undershot water-wheels may utilize the energy for some time during the year, but the difference of level and the cost of the undertaking would be, in most cases, prohibitive.

Power is proportional to the product of the weight of water and the height of the fall. Hence, for a large output of power, either the volume of the water must be large or the height of the fall must be high. In other words, a great fall or a great flow of water is necessary for a large output of power. In Nature the two are seldom found. Hence arises the necessity of *supplementing Nature's gift with the scientific resources of man.* This is best illustrated in hydro-electric projects.

Ordinarily, a medium volume flowing under a moderate head is actually utilized. As the river flows, it meets with many tributary streams, which are not available in the higher altitudes. As the catchment area decreases, the flow of the river becomes smaller and smaller. Hence, the catchment area, which affects the flow of the fall, is also a determining factor, when a fall is to be utilized. Hence, *the flow of the river depends upon*—(1) the **amount,** (2) the **intensity,** (3) the **distribution of the rainfall** over the catchment area. The **catchment** of the river or stream at any point is the area draining into the stream at that point.

Again *the nature of the catchment area* also affects the flow of the stream as—(1) the rainfall may run into the stream; (2) it may be retained in the soil; (3) it may be lost by evaporation, if the surface is large; (4) it may be given up to the vegetation, which is generally the case in forests; (5) it may be diverted to other directions than in the stream which is to be utilized.

A small river had its source in Jaisamundar in Udaipur. A 95 feet high dam was constructed on one side, which stopped

the flow of the stream. The lake is now a very big reservoir having water area of about 21 sq. miles and a catchment area of 691 sq. miles. If the rainfall exceeds a certain point, the excess water goes out through an outlet which forms a small stream on the north of the lake. The *samundar* is not yet utilized, as the State does not find sufficient industry to utilize the power with profit—the lake being at a distance of about 32 miles from Udaipur, and industry has not developed in the territory. The enormous advantage that may accrue to the poor agriculturists by utilizing the fall for power and irrigation, is not appreciated by the State. Thus, it is clear that " A country may have vast rivers and yet very little water-power is capable of economic development." It may not be utilized for want of **demand for power** or **market**, **brain** (human resourcefulness or initiative) or **capital.**

Basis of Water-Power

The basis of water-power is the product of two factors—head and quantity; both are gifts of Nature, although artificial heads may be created and flood water may be stored against the drought of summer. To develop one horse-power a definite quantity of water must fall from one elevation to another in a given time and, in doing so, pass through a turbine.

It must be remembered that *the power-supply must be continuous for most purposes*, and for this a *definite flow of water* must be available throughout the year, and *the fall must be of sufficient amount.* Under the following circumstances, *intermittent power-supply* from hydro-electric plants *may be justified*—(*i*) where the cost of fuel-developed power is very high, or (*ii*) where the seasonal nature of the works, in which the power is to be used, may utilize such power with advantage.

But, if the natural flow or supply of water is seasonal or irregular as in some cases, suitable reservoirs may be constructed to store up the water to get a continuous supply.

(*i*) **Definite flow of water** may be available throughout the year from :—

(*a*) the natural flow of a river or stream, *e.g.*, the Cauvery ;

(*b*) the artificial flow of an irrigation or navigation canal, *e.g.*, the Ganges canal plant at Bahadrabad or Bhola, Amritsar, Ranbir canal, Jammu and Malakand ;

(*c*) a reservoir or lake, *e.g.*, at Naini Tal, Andhra Valley, Khapole, Lena Vala, Bhatghar ;

(*d*) a suitable combination of *a* and *c*—most projects in C. P., cordite factory at Nilgiri's, or *b* and *c*—the Tons River, Rewah.

(*ii*) **Height and Nature of the Fall.**—The nature of the ground determines the head of the fall that may be available.

(1) Natural waterfall, whether of the vertical and spectacular variety, or a cascade, *e.g.*, the Cauvery falls.

(2) A series of rapids not actually forming a waterfall, but may be developed into one suitable for utilization.

(3) Artificial fall created by a lifting dam or barrage in a river or a weir in a canal. The head obtainable will depend upon the height of the dam, and this is determined by the levels of the country above it, which may be flooded, though the ground upon which the foundations will rest and the method for disposing of excess water may affect the practical height, *e.g.*, at Cauvery and Bhandaradara dams.

(4) An artificial fall may be obtained by concentrating the gradual drop of a stream or source over a considerable length, by means of an artificial water channel constructed on a much more gradual slope on the banks or hills above the stream, *e.g.*, at Simla, Darjeeling, Mussoorie, *e.g.* Tata H. E. P. S. Company, most projects in Central Provinces.

(5) An artificial fall created by diverting the waters of a source at a high elevation through the water-parting, *e.g.*, the Pykara Scheme.

Study of Hydro-Electric Scheme

Make a thorough investigation and study all the problems involving geological, meteorological, legal and commercial aspects, etc.; carry out an exhaustive study of the control of the entire main stream showing how the best use can be made of land and water rights, etc.; thoroughly discuss the type of structures for different parts of the hydraulic and electrical works, supports, etc.; discuss the advantage of different sizes, shapes and capacities of plants and structures; present a detailed analysis and give mathematical deductions and tabulated results of every important factor entering into the development; discuss all points and have the report accompanied by an excellent display of maps, and so forth.

Carefully study the following :—

(1) Class of report to be prepared.

(2) Investigation and analysis of all necessary field information including absolute reliability of records, physical character of power site, the foundations, the plan layout.

(3) Plan of the proposed development along the most economical lines, mainly with regard to first cost.

(4) Water supply and its variations, modifications with comparative costs due to storage or pondage.

(5) Estimates of power available and production costs—these should be very liberal and should contain estimates for contingencies sufficient to meet uncertainties as are involved in proper and actual works and cost of auxiliary power. The investigation of the power available and production costs may resolve itself into: (*a*) a complete compilation of all available rainfall and run-off records applicable to the case (make a careful and prolonged investigation of the rainfall and run-off records and see if there is good rainfall and substantial fall within reasonable short distance—the *actual run-off from the catchment area* is the all-important factor); (*b*) a study of the reliability of these records; (*c*) a determination of the most probable run-off for typical years; (*d*) possible modification of flow estimates due to available economical pondage or storage; (*e*) estimates of the power available (with and without storage or pondage); (*f*) a study of the available probable and adequate power market primarily to determine within a moderate distance and plot the load curves; (*g*) determination of the market, nature and distribution of load and reasonably good price for which power can be sold; estimates of financial returns for various operating conditions; (*h*) financial considerations such as the methods and cost of financing the project.

(6) Interference with the rights of others—For example, it may be mentioned that the Mysore Government have a dam across the Cauvery at Kannambadi, about 155 miles upstream, and the question of the amount of water the Mysore Government had a right to store was one of vital importance to a reservoir below.

(7) Whether existing irrigation and mining projects have pre-empted any portion of the stream flow.

(8) Legal difficulties, *e.g.*, inter-statal or inter-national—Examine the laws relating to stream of the State in which the development is to be made. The legal rights, including reference rights, flowage and reservoir rights, rights for way of transmission, and the extent and influences of State and several control on both projects and prices.

(9) Whether there is a large catchment area of the site of the proposed works.

(10) Large areas for impounding flood water.

(11) Good foundation material for a dam to hold the floods.

(12) Low value of land.

(13) Reasonably low cost of the development.

(14) Reasonably low total annual expenditures covering complete operation of the development.

(15) Reasonably cheap terms upon which money is obtained for the development.

(16) The design of the plant, and the structures.

(17) Operation and maintenance.

(18) For the utilization of power, make a careful study of the mineral, forestal, and agricultural possibilities of the surrounding region. These may utilize even flood supplies for seasonal operations, and thus justify a hydro-electric scheme, which will, otherwise, be condemned.

(19) The **probable load factor and diversity factor**.—A high load factor is more essential in the case of hydro-electric installations than in the case of steam or oil plants, the reason being, that while in the latter case some reduction in working expenses is possible by the saving of fuel and lubrication expenses, when owing to reduced load some portion of the plant is not working, no such reduction in working expenses is possible in the case of hydro-electric installations. The possibility of a high load factor should, in this case, be, however, looked for in supplying the surplus power available during the slack hours, at cheap rates, to such industries as cannot afford to pay the full tariff, and not by attempting to increase the peak load for domestic supplies, which will, consequently, leave less power available for the rest of the day. Tube-well irrigation could very well absorb all surplus power available in hours of slack demand at any time of the day or night, and thus raise the load factor to very nearly 100 per cent. Remember that high load factor and low diversity factor are desirable qualities of a load.

(20) A hydro-electric plant costs three times as much as a steam electric plant of the same capacity.

(21) Construction : Construction costs are higher, and operation costs per Kilowatt lower in hydro-electric than steam plants. One of the advantages of power generated by fuel is the low costs of small installations. The cost of power per unit developed is frequently large in such installations, such costs are within reasonable and possible limits of expenses.

Causes of Failure of the Scheme

(1) Most water-power developments are more or less speculative. Frequently the point of view of the promoter, or investor, is too optimistic. The difficulties are underestimated or ignored, and sometimes uninformed public is induced to invest in projects which are undertaken without due consideration or in ignorance of vital factors on which their success depends.

(2) Sometimes power-house is economically placed on sites which are occasionally subjected to flood conditions, as happened at the Phurping Hydro-Electric Works—the plant was partly washed away by the Bagmati River. Perhaps the available information with regard to maximum water levels indicate the safety of the situation and excessive flood, and in some cases an obstruction due to the position of part of the hydraulic work may affect the normal characteristics of the river flow sufficiently to cause high tail water or failure.

(3) When there is a fluctuating source of water or when a reduction in head occurs in years of low rain fall—In the absence of a definite and ever-present fall of at least several feet, no water-power scheme is likely to mature as a commercial success.

(4) Overestimation of the quantity of water available.

(5) Lack of transport facilities to carry materials to the fall —The power-house at Mandi is situated in the interior of the hills and, therefore, it has been necessary to build a railway line to carry the plants and materials to the site of the power-house. The line will not pay its way even if one-third of its cost is charged to the Mandi Scheme.

(6) Failure of a hydraulic equipment or excessive rise of tail water may cause power-house flooding.

(7) Height of back water caused by a dam under different conditions of stream flow may exceed the resulting limits of flowage.

(8) Failure of a section of conduit.

(9) The failure of a forebay or pipe line may destroy both the hill side and power-house below.

(10) Underestimation of the cost of development—The cost of hydro-electric plant is frequently underestimated and is, perhaps, the most common cause of failure, partial or complete. In 1922, when the Electricity Board considered the Mandi Hydro-Electric Scheme, it was represented to cost only 276 lacs, including the cost of the Tramway (now called the Kangra Valley Railway), to yield a return of 16 p. c. with energy sold at six pies per unit. This is now said to cost Rs. 925 lacs as capital and requiring every year further capital to meet the deficit, the sum total of which will be a huge sum of 16·64 crores by 1945 and yielding nothing even with a selling rate of energy fixed at 8·16 pies per unit at 11,000-volt busbars, when the Bahadrabad Scheme sells power to the agriculturists for 7 pies. Only the Government of the Punjab could supply the fund and many private companies would have turned bankrupt for such underestimation.

(11) **High Cost**—Too costly storage or dam is necessary to give a fairly continuous supply to utilize the water.

The total estimated cost of the Mettur Project, which includes the canal system as well as the headworks, is Rs. 7,37,08,000.

Excess cost is often due to :—

(1) Unforeseen difficulties in the physical conditions, entrailing extra cost as constructions. This is well-illustrated in the Ganges Canal Hydro-Electric Scheme, where we find the following:—

"In the time and with the staff available in the early stages, it was impossible to prepare more than brief outline estimates of the probable expenditure leaving the detailed designs to be elaborated as the work progressed and as suitable electrical staff could be recruited for the purpose.

"On the *power station construction some excess expenditure* was caused by the fact that owing to the prevalence of drought conditions it was impossible to close the canal for the periods necessary to construct the under-water works, hence coffer dams and the use of additional quantities of cement had to be resorted to. Extra outlay was also incurred in strengthening the escape bays at the various falls owing to the necessity of passing abnormal supplies during the construction. Experience moreover proved the necessity of installing mechanically operated regulating gates up and down-stream of the stations.

"On the *transmission system additional outlay* has been incurred on the construction of underground cables and overhead line diversions found necessary to approach suitable sub-station sites in the town as fixed in consultation with the licencees after the framing of the earlier estimates. Additional expenditure to the extent of Rs. 1,64,880 was also incurred on the 48 overhead and cable crossings of the various railways to meet the conditions of the revised standard designs for such works.

"Rs. 56,800 has been spent in labour and *compensation on tree-cutting*. All trees within 40 feet of the transmission lines have been cut down affording a large degree of immunity from external interruptions during the heavy wind storms which frequent the locality.

"*Additional sub-stations have* been found necessary in various towns in order to comply with the conditions of the bulk-supply agreements.

"*The original estimate included no provision for operating the three schemes as a connected installation.* The *decision to synchronise* the system involved heavy expenditure on additional equipment and relay switchgear. *A second circuit has also been found*

necessary between Roorkee and Bhola, the conductors for which were not provided for in the original estimate.

"The revised estimate also provides for *the maintenance engineer staff* which has been found necessary during the construction to train the operating staff."*

On the other hand, the work cost at Pykara is £900,000, which is well within the estimated cost and was completed within the scheduled time.

(2) Flood conditions during constructions and consequent extra expenses.

(3) Resulting delays and consequent excessive interest charges.

(4) Large and unexpected expenses for flowage, due to unforeseen backwater conditions.

(5) Over-development, or equipment, not warranted by the market.

(6) Under-development on account of unforeseen no-load factors.

(7) Unexpected cost of transmission due to difficulties in securing the necessary market.

(8) Expensive methods of financing over-development involving over-capitalization, while under-development requires an increased capitalization, which will involve further, and, perhaps, unwarranted, investments.

(9) Want of sufficient demand for power.—Be sure of the market for power, not only of the immediate probable demand, but what the future market may be. Remember that on a poor load factor steam or oil set may often prove superior, while on a good load factor water-power generally has the advantage. An over-estimation of the demand of power is also a vital cause of failure.

(10) Maintenance.—This may involve excessive cost due to unforeseen contingencies. The plant may be poorly designed, foundations may be inadequate or improperly protected.

Proper selection of efficient machinery of suitable capacity. Poor workmanship results in high maintenance cost.

(11) Extent of investigation.—This may be insufficient, and thus it may wreck the whole scheme.

(12) Under-estimation of competition in the sale of energy.

(13) Failure of structure due to poor foundation or other disturbance.

(14) The use of draft tubes of faulty designs and types often account for poor performance of many power plants.

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* Page 25, "The Ganges Canal Hydro-Electric Scheme," by W. L. Stampe, C.I.E., I.S.E.

(15) Faulty design of water passages from the runner to the tail race diminishes the efficiency of the plant. Reduction of efficiency and failure of units to meet the designed capacities, the development of severe vibrations, as also impaired regulation, are among the results of poor draft tube design.

(16) When the fall is at a great distance from centres of industry.

The sources of losses are mostly from (1) conservation of rain-water from the time of rainfall up to, and including, the point of storage, and the entrance through the strainers or screens at the intake of the developed waterways ; also the accurate determination of the economical development with reference to the available water-power ; (2) turbines complete ; (3) transmission lines ; (4) feeders ; (5) electric distribution service ; and (6) the open conduit (unlined) and the pipe line.

The **objects** should be to secure :

(*a*) the highest possible practical use of a given quantity of rain-water;
(*b*) the best and most economical location of the development sites;
(*c*) the most economical development as a whole; and
(*d*) the very best and most economical system of operating conditions.

When a project involves all of the fundamental considerations in the advantageous manner mentioned above and the sources of losses are cut to the minimum, the success of the development is beyond dispute.

Selection of Water-Power Site

Choice of water-power site should be based upon the following considerations :—

(1) Whether the most natural drop or fall on a portion or reach of the stream, where there is a well-defined slope, is available.

(2) Relative facility and cost of construction.

(3) The best position for the dam, conduit, forebay, pipe line, power house, and tail race.

Note that the location, arrangement, and cost of the headworks are governed almost entirely by natural conditions. The best location of the forebay is usually a matter of cost, but with favourable geologic and topographic conditions, the most effective combination of forebay-regulating basin should be made at relatively low cost. The location of the pipe line requires hill side, good slope, and relatively cheap and ample means for supports and

anchorage, etc. The location of a conduit (non-pressure type) depends essentially on location conditions. Power-house location demands good foundation and the best use of head to give maximum power and efficiency at all stages of the river.

(4) Most favourable geological and topographical conditions.

(5) Best position for storage—safe reservoir bottom and head, and least total cost.

(6) The continuity of supply of available power.

(7) A market for power.

(8) Least possible hydraulic losses, due to leakage, seepage, evaporation, roughness factor, tail water level and backwater conditions, conduit, pipe line and turbine.

Thus a reliable analysis of (1) hydrographic and hydrometric data conditions, and (2) a reliable estimate of the load factor and character of the load must be determined for the selection of the site.

Selection when more than one site is available (*e.g.*, in the Mandi *vs.* Madhopur, Sutlej, etc., Scheme).—The leading factors governing the choice of the project are as follows :—

(*a*) The most reliable records of stream flow.

(*b*) Total amount of energy which can be generated at a reasonable cost and the amount of power should be most nearly equal to that desired.

(*c*) Amount of capital expenditure to be incurred before power can be made available, *viz.*, (the amount required for the completion of the first stage of development), and the minimum cost of development per kW. harnessed.

(*d*) Ultimate cost of the whole scheme, and the cost per kW. harnessed.

(*e*) Adaptability of the scheme for development by easy stages, so that power can be developed as the demand increases, and thus the capital does not remain idle pending the creation of a market for surplus power.

(*f*) Rights of pre-emption in favour of irrigation, mining, etc.

(*g*) Period of completion of the first stage of development.—Any scheme which can be completed earlier than the rest affords a direct saving in charges for interest and cost of power to the existing consumers. Hence, the minimum unproductive period of construction is an important point for consideration.

(*h*) Comparative freedom from risks due to abnormal seasons, other physical changes, and generally causes beyond human control.

(*i*) Rate per unit (Kilowatt hour), which would, in all cases, just cover $11\frac{1}{2}$ per cent. on capital cost (6 per cent. for interest,

$2\frac{1}{2}$ per cent. for depreciation, and 3 per cent. for maintenance and working expenses) or give a predetermined profit.

(*j*) The shortest transmission line with its consequent lowest cost and least liability to interruption of service.

(*k*) The greatest possible future increase in output.

(*l*) The greatest possible future increase in output due to possible future storage.

(*m*) The greatest opportunity for additional development close by, tending to centralize operation and effect reduction in the required number of spare parts, machine-shop equipment, etc.

(*n*) The most accessible site with transport facilities.

(*o*) Proximity to market.

Illustration of "Selection"

* Let us apply the principles categorically to Mandi *vs.* Madhopur, Sutlej, etc., Schemes, and examine the feasibility of the projects.

The Uhl (Mandi) Scheme.—This scheme contemplates the harnessing of the Uhl, a tributary of the Beas. In the first stage of its development it is proposed to erect a low weir across its natural channel, and divert the water through a tunnel about $2\frac{1}{2}$ miles long into steel flume pipes leading to a power house about 1,800 ft. below. The catchment area is 141·6 sq. miles. The minimum discharge *recorded* during the last three years under observation is $98\frac{1}{2}$ cusecs, while the maximum during the summer months exceeds 1,000 cusecs. For the first stage the discharge has been taken at $112\frac{1}{2}$ cusecs.

In the second stage it is proposed to erect a dam 270 ft. high, so that an average discharge of 225 cusecs can be obtained throughout the year by storage of monsoon water.

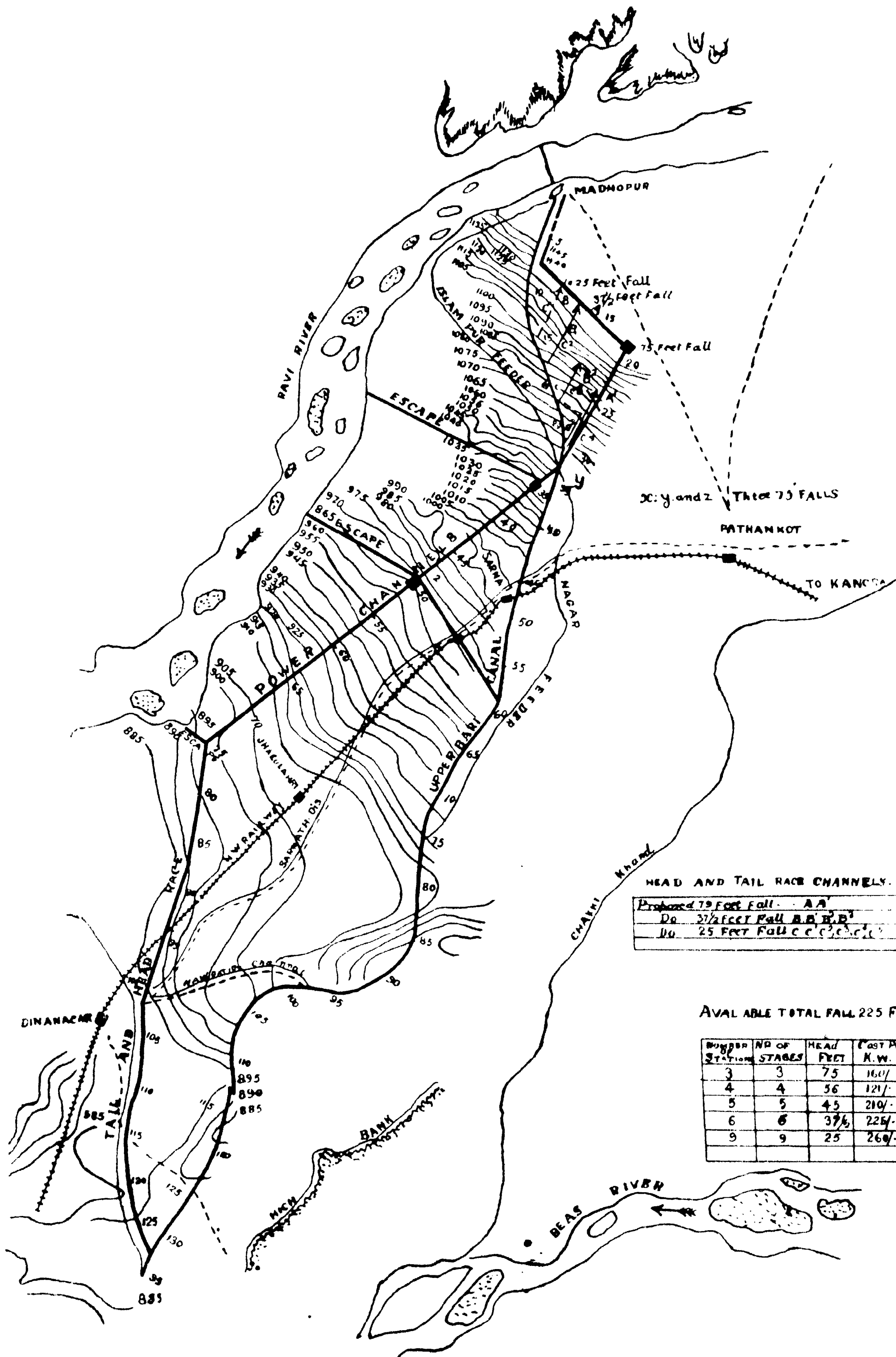
In the third stage a further fall of 1,200 ft. is proposed to be utilized by erecting a second power house below the tail race of the first.

(*a*) **Continuity of supply of water—**

† "The first thing is that so far as the discharge of water is concerned, the Madhopur Scheme takes the palm." "The water discharge of Uhl stream depends a great deal on the weather, the catchment area being to a very large extent snow-bound. The minimum discharge of $29\frac{1}{2}$ cusecs was *observed* in January, 1923. This was, by no means, the severest winter on record, etc. The

* Parts adopted from "A brief discussion on the comparative merits of different schemes," by Rai Bahadur Rala Ram, C. I. E., I. S. O., M. I. E. (Ind.), Retired Chief Engineer of the Indian State Railways, and the Punjab Council Debate.

† Prof. Ruchi Ram Sahani, in the Punjab Council.



Number of Stations	No of Stages	Head Feet	Cost Per K.W.
3	3	75	160/
4	4	56	121/-
5	5	45	210/-
6	6	37½	225/-
9	9	25	260/-

minimum discharge of the Ravi on which the Madhopur Scheme is dependant is 1850 cusecs at the time. The minimum water discharge for the Madhopur Scheme would be at 1,314 cusecs, while the minimum discharge has been found after an examination of the figures for three years for the Mandi Scheme to be 29 cusecs." "Even during these three years there was one day, I believe it was in January, 1923, when the discharge of the water fell to 29 cusecs though the minimum, upon which the scheme is based, is 112 cusecs. It was a very cold day and so the water froze and the discharge fell to 29 cusecs. I have counted at least 141 days during the last 15 months when the discharge fell below the minimum given by the experts who framed the scheme." "No doubt the Mandi Scheme has not been examined for more than three years, while the Madhopur Scheme has been examined for about 25 years. This is the result of such an examination. Then again on information it is believed to be correct and reliable, although the minimum discharge, as placed before the experts in England, was stated to be 112 cusecs; as a matter of fact, the discharge for 141 days during that period was below 112 cusecs. This is a remarkable superiority that the Madhopur Scheme has over the Mandi Scheme, because the creation of energy, after all, depends upon the discharge of water."

"The discharges in the case of Madhopur and Sutlej, as taken above for purpose of comparison, are the minimum recorded during the period for which Uhl discharges are available. No doubt the records of Madhopur and the Sutlej show that the Ravi and the Sutlej have had minimum discharges of 1,300 and 2,700 cusecs respectively; but a similar reduction would occur in the case of Uhl, if discharges for this stream had been available for the same number of years, as the comparative daily discharge diagrams of the 5 main rivers of the Punjab along with those of the Jamna and Uhl show that the seasonal fluctuations pretty well synchronise with each other.

(*b*) **Comparison with regard to ultimate output**—

"With regard to the total output at the final stage, the different schemes compare as below:—

(1) Uhl (Mandi) ... ... 45,000 kW.
(2) Madhopur (Ravi) ... ... 55,500 kW.
(3) Sutlej ... ... 102,800 kW.
(4) Rasul (Jhelum) ... ... 21,000 kW.

"In the case of Uhl, to obtain an average discharge of 225 cusecs throughout the year, a dam, 270 feet high, estimated to cost Rs. 1,00,00,000, will have to be built for storage of water during the rains to make up for the deficiency during the cold

weather ; and that, in the event of a severe earthquake like that which occurred in this locality in 1905, not only this high dam, but the works connected with the whole installation, are liable to be damaged.

(*c*) **Cost**

Table showing comparison between the Mandi and the Madhopur Schemes in the first stage.

Particulars	Output kW.	Capital cost in lakhs of rupees			Capital cost per kW. Rs.
		Production	Trans-mission.	Total.	
Uhl	13,200	245	215	460	3,485
Madhopur (Ravi) (without steam help)	9,250	80	85	165	1,784
(*b*) with 10 % steam help	17,000	110	170	280	1,647
(*c*) with $20\frac{1}{2}$ % steam help	34,000	140	200	340	1,000

" In the case of Sutlej and Rasul the development cannot be made in stages. The comparison might, therefore, be made with the final stage of development only. The table below shows how the different projects compare.

" In the first stage of development the Mandi and the Madhopur Schemes compare as stated above.

Table showing comparison between the different schemes in the final stage of development.

Particulars.	Output kW.	Capital cost in lakhs of rupees.			Capital cost per Kilowatt.
		Production.	Trans-mission	Total.	Rs
Uhl—(Mandi) ...	44,400	Details not known.	...	1,206	2,716
Madhopur— ...					
Ravi—					
(*a*) without steam help	27,750	235	200	435	1,568
(*b*) with 10 per cent. steam help.	51,000	290	300	590	1,157
(*c*) with $26\frac{1}{2}$ per cent. steam help.	102,000	380	450	830	814
Sutlej—					
(*a*) without steam help discharge 3,300 cusecs.	80,000	569	450	1,019	1,274
(*b*) with $12\frac{1}{2}$ per cent steam help.	160,000	900	600	1,500	938
Rasul—					
Requires 22 per cent. steam help	21,000	102	150	252	1,200

COST PER UNIT KILOWATT HOUR.

The following tabulated statement shows the cost per unit in the first and final stages in each case.

Particulars	Output	Capital cost: Total cost in lakhs of rupees.	Capital cost: Per kW. of output	Charge per kW hour at consumer's terminals: 11⅓ °/o per annum on capital cost.	Charge per kW hour at consumer's terminals: Extra for coal.	Charge per kW hour at consumer's terminals: Total	Remarks.
1st Stage.	kW.		Rs	Pies	Pies	Pies	
Uhl (Mandi) ...	13,200	460	3,485	8·78	...	8·78*	* Excluding royalty payable to Mandi State
Madhopur (Ravi)—							
(*a*) without steam help ...	9,250	165	1,784	4·50		4·50	
(*b*) with 10 °/o steam help ..	17,000	280	1,647	4·14	·55	4·69	
(*c*) with 26½ °/o steam help ...	34,000	340	1,000	2·52	1·46	3·98	
Final Stage.							
Uhl (Mandi) ...	44,400	1,206	2,716	6·85	...	6·85*	
Madhopur (Ravi)—							
(*a*) with 10 °/o steam help ...	51,000	590	1,157	2·92	·55	3·47	
(*b*) with 26½ °/o steam help...	102,000	830	814	2·05	2·46	3·51	
Sutlej—							
(*a*) without steam help ...	80,000	1,019	1,274	3·22	...	3·22	
(*b*) with 12½ °/o steam help ...	160,000	1,500	938	2·36	·69	3·05	
Rasul—							
With 22 °/o steam help ...	21,000	252	1,200	3·02	1·21	4·23	
Madhopur-Cum-Rasul—							
With 26½ °/o and 22 °/o steam help respectively ...	55,000	550	1,000	2·52	1 37	3·89	

"From the above tabulated statement, it is clear that Madhopur, with 26½ per cent. steam help, gives the best results both in the first stage of development and in the final stage when the whole scheme is fully developed. The Sutlej, with 12½ per cent. steam help, comes next, while the Uhl is the most expensive scheme of the whole lot.

"With regard to the minimum amount of capital required to be invested before any power can be made available at all, Madhopur again shows the best result. Of course, Rasul compares quite favourably with Madhopur as it requires a capital of Rs. 252 lacs only for generating 21,000 kW. against a capital of Rs. 280 lacs for generating 17,000 kW. at Madhopur with 10 per cent. steam help.

(*d*) **Ultimate cost.**

"The total cost of the Madhopur Scheme, according to the figures in the first stage, would be Rs. 1,80,00,000, as compared with the official figures for the Mandi Scheme, which are Rs. 5,26,20,000 not including the Rs. 71,00,000 which would be spent on the railway, and which, the department considers, to be paying. Probably, such a railway would not be self-supporting, and if Rs. 71,00,000 is also included, these figures would stand at Rs. 1,80,00,000 for the Madhopur Scheme, as compared with Rs. 5,97,00,000 for the Mandi Scheme. This is also a great disparity. *Vide* also "c" and "e" above.

(*e*) **Adaptability for Construction by Stages.**

"From this point of view, the Madhopur Scheme is by far the best of the lot. As shown in the Table, p. 22, it can start with a capital of Rs. 165 lakhs, yielding an output of 9,250 kW. with 10 per cent. steam help, the capacity can be increased to 17,000 kW. with an additional capital expenditure of 115 lakhs only, while a further addition of 30 lakhs for plant and 30 lakhs for transmission line (for utilization of additional power) the capacity can be brought up to 34,000 kW. In the case of Uhl the expenditure of at least Rs. 460 lacs is necessary for production of 13,200 kW. Also in addition to this, Rs. 71 lakhs must be spent on the construction of the Kangra Valley Railway line. The next stage of development requires a further capital expenditure of Rs. 144 lakhs.

Rasul and Sutlej cannot be taken up in stages at all; but as the total output of Rasul is only 21,000 kW., the objection regarding capital lying idle for any great length of time, hardly applies in this case.

From the above statement it is clear that the Sutlej, with steam help, would supply the cheapest power to the consumer. Even without steam help, it is cheaper than the rest. The difficulty, however, will be that for many years there will not be a market for the entire output, with the result that a much larger rate will have to be charged for the actual amount of power taken by the consumer, and until such time as the demand increases to the full capacity of 80,000 kW. or thereabouts. So long as the demand remains below 50,000 kW., Madhopur will decidedly be able to supply power at a much cheaper rate than the Sutlej.

The Madhopur Scheme is cheaper than the Mandi Scheme. It will cost 2½ crores in three stages, while the Mandi Scheme will cost more than 5 crores in the first stage, and 12 crores, when it is completed, in three stages. The soundness of the Madhopur

Scheme is obvious from the fact that the organisers offer current for sale at 33 per cent. less than the cost price of the Mandi Scheme current.

The Madhopur Scheme is capable of development by five stages.

(*f*) " In order that the **scheme may not, in any way, interfere with irrigation**, it is proposed to join the canal with each of the power channels, and providing in-lets therefrom into the canal; similarly, escapes from each of the three tail race channels are provided for leading the water back into the river. This will enable not only the power stations being worked by escaping surplus water into the river, when none is required lower down the canal, but it will also permit independent working of each power station, while, of course, there will be no interference with the running of the canal, when any of the power station is under repairs, or is temporarily out of commission for any reason whatsoever.

" When the 102,000 kW. obtained, as detailed above, have been utilized, a further development is possible on the main river above the Madhopur head works. Between Madhopur and Kukesar (a distance of 9 miles), the bed slope of the Ravi amounts to 160 ft. Near Mukesar the river is confined between rocky banks, where it would not be difficult to build a dam (less than 1000 ft. in length) and about 80 ft. high, so as to obtain a further total fall of 225 ft., which (with steam help for winter months) will yield a further supply of 102,000 kW.

" The site of the first power house of this scheme is only 4½ miles from Pathankot, and a broad gauge railway siding could be provided with second-hand permanent way at a very cheap cost, as the line will practically be a surface one without any drainage to cross.

(*g*) **Time for completion.**

" Regarding the time for completion, Rasul would head the list, as it involves the least amount of work. It is estimated that Rasul could be completed in 2 years' time. The first stage of Madhopur would probably take 3 years. The plant for 10 per cent. steam help is already available in the existing steam plants installed, or under installation, within the area of consumption. Uhl and Sutlej will not take less than 5 years; consequently, the advantage from this point of view also lies with Rasul and Madhopur.

(*h*) **Comparative Freedom from Risks.**

" It goes without saying that Madhopur and Rasul being both situated in the plains, are comparatively immune against

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risks due to earthquakes and landslips. The same cannot be said of the Uhl where the earthquake of 1905 caused such a havoc. The tunnel, the railway line, the main transmission line and the power house of the Mandi Scheme are all located within the earthquake zone. The landslips would be another source of ever-recurring danger in the case of Uhl, as the transmission lines from the power house up to the point where they debouch into the plains will lie in hilly country. The dislocation due to interruption caused by landslips would seriously affect the entire area of consumption."

"The storage lake will be silted up and will soon be filled up with debris and boulders brought down from the very steep slopes in the catchment area owing to practically still water condition of the lake."

"The site is in region which is liable to seismic disturbances." "Under the heaviest flood conditions considerable travel of boulders and heavy debris may be expected in the river bed and through the diversion tunnel where it is formed." "When the greatest transport of debris is taking place the tunnel will not serve to pass the material as it will be stopped in the still water above the tunnel."—— "As the travel of dammed water rises during a flood so will the place of deposit move up-stream and where the water begins to cover the intake works, heavy deposit of material may be expected at the weir." The point for serious consideration, therefore, is as to whether obstructions may not be deposited at the tunnel gates, and whether the high flood state of the river at the times the gates require to be shut may not hinder the removal of obstructions. If a gate cannot be shut home, much of the storage would probably be lost."

Some uncertainties exist, mainly owing to the short time during which observations have been possible as to the actual maximum and minimum discharges of the Uhl River and as to its debris carrying capacity.

In the opinion of Messrs. Sir Alexander Gibb and partners, Consulting Engineers, London, the storage lake will be silted up, and will soon be filled up with debris and boulders brought down from the very steep slopes in the catchment area owing to practically still water condition of the lake; and Messrs. Merz and McLellan's report strongly suggest the deposition of debris and silting up of the reservoir with consequent loss of storage capacity.

* Messrs. Sir Alexander Gibb and Partners.
Vide page 1 of Mirz and McLellan's Report.

"Take the Mandi Scheme from another point of view: It lies in very high regions, and once or twice at least in the year the small *nalla* which is going to be the water-supply in the scheme is practically frozen. In order to obviate the difficulty, it is contemplated that a big dam some 270 feet high should be constructed at an elevation of six thousand feet from the sea-level. This seems to be obviously a risk on which even a layman could venture to give an opinion. To construct a dam some 270 feet high at a height of six thousand feet above sea-level which would practically keep in check water to the extent of 930 millions cubic feet seems to be extremely risky even from a layman's point of view.

"**About the silt**:—It is well-known that if the critical velocity ratio exceeds a certain point, then the scouring process goes on; if it falls below unit, then the silting process is started. In the Madhopur Scheme silting cannot take place here according to Mr. R. K. Kennedy's formula (Chief Engineer, Irrigation Department). On the other hand, silting will take place, if we go into the calculations, in the dam which is proposed to be constructed under the Mandi Scheme.

(*j*) The site at Mandi is difficult to approach, and before the scheme can be completed, a narrow gauge railway line about 100 miles long will have to be built from Pathankot to the site of the works, in addition to a tramway for haulage of plant.

The power generated will have to be transmitted across hilly country; an extra length of transmission line will be required about 75 miles in the first stage, while in the second stage it is proposed to provide an additional 100 miles (approximately) from tho power house to Kalka. To prevent corona losses at higher altitudes thicker cables and longer arms will have to be provided, which will make this part of transmission line comparatively more expensive.

(*k*) "The energy which the framers of the Madhopur Scheme promise us is about three times as great as the total energy from the Mandi Scheme, the exact figures being 107,000 Kilowatts, while the Mandi Scheme promises us only 33,000 Kilowatts.

(*l*) "What is proposed to be done, in the case of Uhl, by means of seasonal storage, can be accomplished in the case of Madhopur and Sutlej by auxiliary steam help, and without any risk due to fluctuations of rainfall, or occurrence of a severe earthquake. With steam help, averaging 10 per cent., Madhopur can be harnessed for 3,400 cusecs increasing the output to 102,000 kW., while with similar steam help of 12½ per cent., the Sutlej would produce about 160,000 kW.

(*m*) "The total fall at Madhopur is taken at 450 ft. This is made up of 2 items, *viz.*, the fall of 225 ft. available in the canal, and an additional fall of 225 ft. in the Ravi river at a distance of about 9 miles above the headworks.

"With regard to the total output of energy, the Uhl Scheme is far inferior to the Sutlej without any steam help at all, while the Madhopur Scheme, with an average steam help of 10 per cent., can yield more than double the output of the former. If steam help were increased to 26½ per cent., Ravi can yield 204,000 kW., which is more than 4½ times the ultimate capacity of Uhl.

(*n*) "So far as accessibility is concerned, there is no comparison at all between the two schemes. Madhopur is, in the plains, practically in the heart of a well-populated country, being very near to the factories which might exploit the energy when it is available, while Mandi is situated far way from civilisation, if I may so put it, without any disrespect to that State. Mandi is situated in an out of the way and isolated part of the country, and the energy produced there would not be readily accessible to people who want to consume it."

(*o*) The distance from the market being, in the case of Madhopur, practically nil ; while in the case of Mandi, at least 100 miles, if we take the distance between Dhariwal and Mandi State.

Summary

To sum up : In the foregoing discussion, it has been shown that the Madhopur Scheme, combined with Rasul, is far superior to the Uhl Scheme from every point of view. For convenience of reference, the comparative advantages and disadvantages of the two schemes are summarised below : –

		Madhopur-cum-Rasul.	Uhl.
(1) Greater Output	Ist stage ...	55,000 kW. ...	13,200 kW.
	Final stage ...	225,000 kW. ...	44,400 kW.
(2) Cheaper to harness (cost per kW. harnessed).	Ist stage ...	Rs. 1,000 ...	Rs. 3,485
	Final stage ..	Rs. 759 ...	Rs. 2,716
(3) Cost per unit (kWh.)	Ist stage ...	3·89 pies ...	8·78 pies
	Final stage...	3·35 pies ...	6·88 pies.

	Madhopur-cum-Rasul.	Uhl.
(4) Harnessing by stages ...	6 stages ...	2 stages.
(5) Period of completion of 1st power house.	2 to 3 years ...	4 to 5 years.
(6) Comparative freedom from risks	No risk ...	Liable to risk from severe earthquake and landslips
(7) Accessibility	Within 5 miles from broad gauge Ry. in plains.	Requires about 100 miles of narrow gauge hill railway.
(8) Proximity to centre of market ...	70 miles from Amritsar.	145 miles from Amritsar.
(9) Catchment area ...	3,000 sq. miles from Ravi alone, contribution of Jhelum in addition to this.	141·6 sq miles
(10) Liability of fluctuations in discharges.	Less danger from severe winter.	Severe winter might seriously affect discharges.
(11) Royalty ...	Any royalty charged will benefit provincial finances.	Royalty of $4\frac{1}{2}$ lakhs will have to be paid to a foreign state.
(12) Reliability of data or otherwise ...	No uncertain factor beyond the ordinary minor errors of estimating. Daily discharge observations available for 65 years.	Several uncertain factors such as:— (*a*) Discharge based on 3 years' observations only ; (*b*) difficulty of correctly estimating in case of tunnel and hill railway, etc.

Net Result

"The net result of the considerations advanced above is that by accepting the Madhopur-cum-Rasul Scheme, the hydro-electric energy would be made available 2 years earlier than that of the (Uhl-Mandi) project. There will be, at the same time, a saving of as much as 4·2 crores of rupees in capital expenditure, while the output will be 2·8 times greater. The current will be obtained at a rate which will be 5·37 pies lower in the first stage and 3·34 pies in the final stage per unit. This means, in effect, that there will be a net saving of $\left(\frac{44\,400\text{kW.} \times 365 \text{ days} \times 24 \text{ hours} \times 3{\cdot}34 \text{ pies}}{192 \text{ pies}}\right)$ $=67{\cdot}66$ lakhs of rupees every year, and in addition to this, 4·4 lakhs of royalty which will be paid to Mandi State. This amount,

if capitalized at 5 per cent., will come up to 14½ crores, and this, when added to the sum of 4·2 crores of saving in the initial capital cost, will make the total saving of rupees 18½ crores."

Conservation of Natural Fuel Rosources

Before the hydro-electric project is taken in hand, a comparative idea of the cost of power from various sources, *e.g.*, coal, oil, gas, and air, must be carefully studied, and it would be seen if the saving in fuel and high load factor in the hydro-electric scheme are sufficient to justify the extra outlay for development. Only then it will be undertaken, otherwise not. Thus it will be clear while Jammu, Mussorie, Naini Tal, Cauvery and Tata schemes fulfil our expectation of cheap supply of power in comparison with coal fuel plants, the Mandi and the Ganges canal hydro-electric schemes do not stand in competition with coal fuel plants. Some idea of the comparative merits of the schemes, stated hereunder, may be studied with profit.

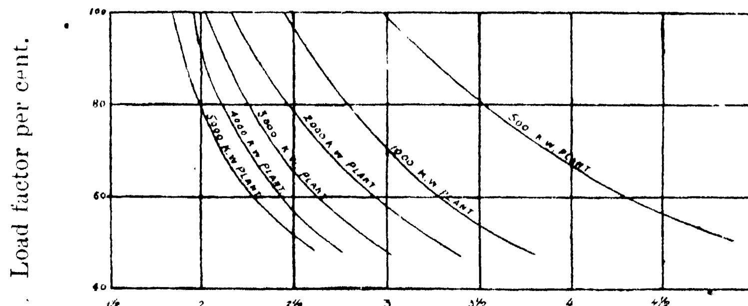
In order to be commercially successful, the cost of power from a hydro-electric installation must not be more than the cost for the power from the natural fuels—coal, oil, or gas, and the capital cost must not be prohibitive. The chief competitor of water-power is steam, which, in modern power plants, is able to give bulk supply very economically. In any plant, the total cost is made up of the capital cost and the running charges. The capital charges vary with local circumstances and physical characteristics of the sites, and hence the interest, depreciation, sinking fund charges. The capital cost of a hydro-electric plant is invariably greater than that of a steam or oil engine plant, unless the available head is great and no artificial storage is necessary; for the former must be developed at the place where it is found, usually out of way places away from centres of population, difficult of access, and the transportation charges for the materials of construction are heavy. Dams, headworks, and pipe lines often prove very costly items, and so is the cost of long transmission lines necessary for conveying the power to the market place where it is to be utilized; whereas the latter may be located in the most favourable place with respect to the market for power and to which the fuel required may be brought much more cheaply as compared with the cost of long distance transmission. Of the running charges, many items, such as stores and supplies, repairs and depreciation, staff, office, and legal expenses, are common to both. The point, where the hydro-electric plant chiefly scores over the fuel plant, is the cost of fuel, which is nil; whereas

the quantity of fuel consumed in a fuel plant increases with the power generated, although the weight of fuel consumed per unit diminishes as the total power generated increases. See figs. I and II.*

Fig. I. Station capacity H. P.
Coal consumption in central stations

LOAD FACTOR %
H.P HOURS PER 100,000 B.Th.U.
COAL CONSUMPTION IN CENTRAL STATIONS IN AMERICA (H.E.P.C. Ontario)
COAL POUNDS PER H.P. HOUR.
0 20000 40000 60000 80000 100,000 120,000 140,000 160,000 180,000 200,
STATION CAPACITY H.P

Fig. II. Coal pounds per unit (kWh.)
Coal consumption according to plant size and load factor (H. E. P. C. Ontario).



* Curves are reproduced from the report of the Hydro-Electro Power Commission of Ontario

Fig. I shows the average number of pounds of coal used per horse power, hour in America, the horse power hours obtained per 100,000 British Thermal units, and the load factors of the particular plants from which these curves were derived.

Fig. II shows the number of pounds of coal consumed per unit (kWh.) according to the size of the plant and the load factor based also on American data.

But the steam plant can always be constructed of the size proportional to its prospective market, and can be increased and enlarged, as its market demands, to any extent and without over-investment.

Thus, in a given hydro-electric plant, the cost of power production is a fixed definite sum per annum, no matter what the number of units generated; and consequently, the cost per unit generated diminishes as the number of units generated increases; in the case of fuel plant, the total cost rises as the number of units generated increases. Therefore, whether any hydro-electric scheme will prove successful or not, depends on the cost of fuel as delivered in the locality where power is wanted, and whether the capital charges for interest and depreciation on the excess cost of the undertaking is less or greater than this cost of fuel used by the fuel plant. Approximate price of coal in India is given below.

For example, suppose a plant of 10,000 kW. capacity is to be installed within a reasonable transmission distance from a waterfall capable of delivering the power required. The capital cost of the undertaking is Rs. 800 per kW. of the installed capacity. There is also a suitable site available at the centre of the power supply for the installation of a steam plant whose capital cost is Rs. 450 per kW. of the installed capacity. Allowing 3000 kW. of the plant capacity set apart for the house supply (*i.e.*, for the installation itself) and the reserve, find the cost of coal per ton required for the steam plant which will just balance the extra high cost of the hydro-electric plant at a rate of 12% for interest and depreciation, if the load factor is (*i*) 90%, (*ii*) 30%. Allow 2 lbs. of coal per kWh.

Extra capital cost of the hydro-electric plant over that of the steam plant = Rs. 10,000 (800—450) = Rs. 35,00,000.
Therefore, excess of interest and depreciation on this amount at 12% per annum = Rs. $\frac{35.00,000 \times 12}{100}$ = Rs. 4,20,000, and this must be balanced by the cost of coal required.

(*i*) Units delivered at 90% load factor = (10,000 − 3,00) × 365 × 24 × 0·9 = 55,188,000 kWh per annum.

(*ii*) Units delivered at 30% load factor = (10,000 − 3,000) × (365 × 24 × 0·3) = 1,83,96,000 kWh. per annum.

∴ Weight of coal required in the first case for the power delivered = $\frac{55,188,000 \times 2}{2,240}$ tons = 49,275 tons/annum.

Whereas weight of coal required in the 2nd case = $\frac{49,275}{3}$ = 16,425 tons/annum.

Weight of coal required for home use = $\frac{3,000 \times 365 \times 24 \times 2}{2,240}$ = 23,464 tons/annum.

Therefore, with a low load factor, as in the second case, everything else being the same, the steam plant will be more economical if the cost of coal is less than Rs. $\frac{4,20,000}{16,425 + 23,464}$ = Rs. 10-8-0 per ton; whilst with a high load factor, as in the first case, the cost of coal will have to be less than Rs. $\frac{4,20,000}{49,275 + 23,464}$ = Rs. 5-12 per ton in order to be able to compete with the hydro-electric plant.

Obviously, not only the cost of coal, but also the load factor, is of very great importance. The chances for the success of the hydro-electric plant increases with the load factor.

When the practicability and feasibility of the scheme have been favourable, we have to see the cost of the scheme and compare it with the cost of energy from a steam or diesel plant, but as it is unlikely that even the superior thermal efficiency of the diesel engine will enable it to compete with steam turbine plant of any size due to the higher cost of liquid fuel, we may, therefore, compare hydro-electric plant only with a steam plant. For this we should know (1) the fuel consumed in power house, and the quantity of coal consumed per unit (Note that this depends upon the capacity of the plant and the size of the unit, whereas in a hydro-electric plant there is no such difference); (2) the cost of coal is different in different parts of India; (3) the cost of unit of power as practically found in different stations; (4) the cost of power in other hydro-electric power station in the vicinity.

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Approximate price of coal in India, taking cost of 1st class coal in the Bengal coal field as Rs. 3 per ton.

Reference number on recorded map.	Area.	Fuel referred to	1933 cost per ton on basis stated. Rs	as.
1	Chittagong area ...	Bengal coal by sea ...	13	0
2	Bengal coal fields and surrounding district (Asansol).	Bengal coal ...	3	0
3	Bihar and United Provinces (Sone)	Do. ...	7	0
4	United Provinces (Cawnpore) ...	Do. ...	9	8
5	United Provinces and Punjab (Saharanpur).	Do. ...	11	8
6	Punjab (Lahore) ...	Do. ...	13	0
	Ditto ...	Punjab coal ...	9	0
7	Punjab and North-West Frontier Province (Peshawar).	Bengal coal ...	15	0
	Ditto ...	Punjab coal ..	8	8
8	Punjab (Multan) ...	Bengal coal ...	14	8
	Ditto ...	Punjab coal ...	10	0
9	Baluchistan (Quetta) ...	Bengal coal ..	17	0
	Ditto ...	Local coal ..	7	0
10	Sind (Sukkur) ...	Bengal coal ...	15	8
	Ditto ...	Baluchistan coal ...	10	0
11	Sind (Karachi) ...	Bengal coal by sea ...	14	0
	Ditto ...	Bengal coal by rail ...	16	0
12	Rajputana and Central India (Kotah).	Bengal coal ...	12	0
13	Central India and C. P. (Katni) ...	Bengal coal ...	9	8
	Ditto ...	Umaria coal ...	4	0
14	C. P. (Bilaspur) ..	Bengal coal ...	8	8
	Ditto ...	Umaria coal ...	6	8
15	Gujerat (Morvi) ...	Bengal coal ...	14	12
16	Central Provinces (Nagpur) ...	Bengal coal ...	10	0
	Ditto ...	Pench coal ...	5	8
	Ditto ...	Bellarpur coal ...	6	8
17	Northern Bombay (Bombay) ...	Bengal coal by sea ...	14	0
	Ditto ...	Bengal coal by rail ...	15	8
18	Hyderabad (Itarsi) ...	Bengal coal ...	16	12
	Ditto ...	Singareni coal ...	8	0
19	Hyderabad (Singareni) ...	Bengal coal ...	15	0
	Ditto ...	Singareni coal ...	6	0
20	Northern Madras (Vizgapatam) ...	Bengal coal by sea ...	11	0
	Ditto ...	Bengal coal by rail ...	10	0
	Ditto ...	Singareni coal ...	10	0

Reference number on recorded map.	Area.	Fuel referred to.	1933 cost per ton on basis stated.
			Rs. as.
21	Southern Bombay and Mysore (Mormagao).	Bengal coal by sea ...	13 0
	Ditto ...	Bengal coal by rail ...	22 0
	Ditto ...	Singareni coal ...	10 0
22	Southern Madras (Madras) ...	Bengal coal by sea ...	11 0
	Ditto ...	Bengal coal by rail ...	14 0
	Ditto ...	Singareni coal ...	10 0
23	Lower Burma (Rangoon) ...	Bengal coal by sea ...	14 0
24	E. Bengal and S Assam (Mymensingh).	Local coal ...	5 0
25	Northern Assam (Dibrugarh) ...	Assam coal ...	7 0

*"**Cost of Hydro-Electric Power.**—It will be studied with interest that the average of capital cost in Sweden is £ 1-7 per turbine horse-power, and when grouped according to size of the installation, the average cost ranges from £ 27 for installation of less than 200 h. p. to £ 6-8 for an installation of 20,000 h. p. In Scotland on a pre-War scale it is £ 25 per installed turbine horse-power. The cost of power in bulk at Niagara Falls is 9 dollars per horse-power year ; this is increased by transformation and transmission charges to 11·5 dollars, at the falls to 14 dollars at a distance of 54 miles and to 38 dollars at a distance to 237 miles.

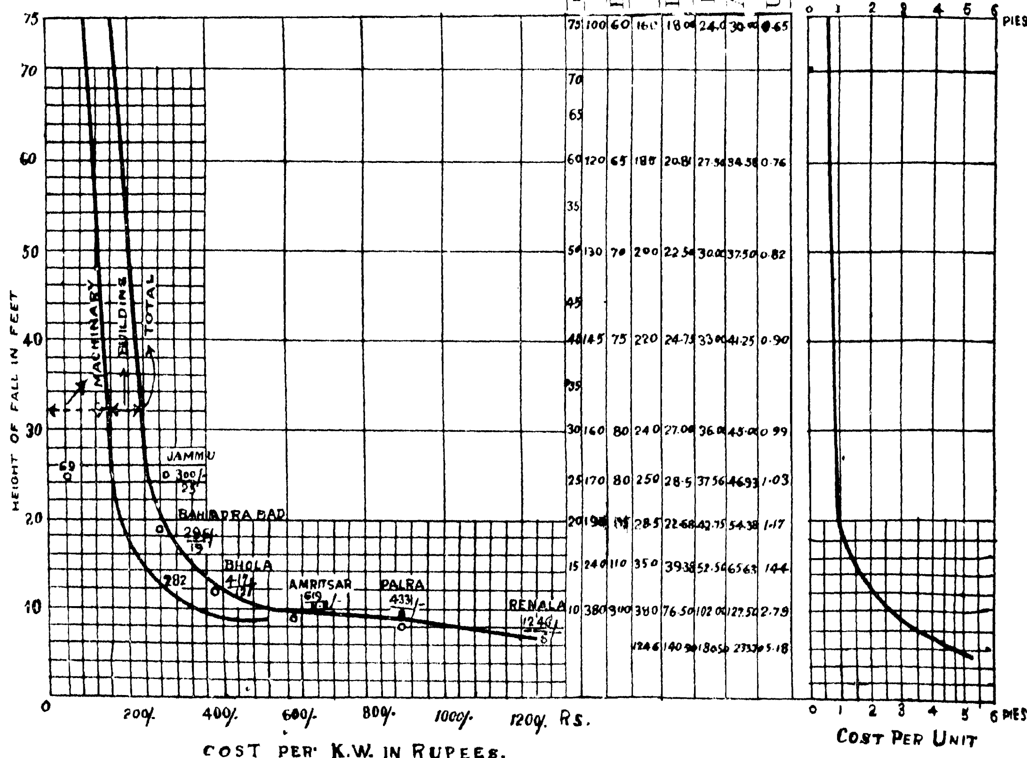
"An examination of some 120 European installations shows that for large installations of upwards of 10,000 electric horse-power the minimum cost of the hydraulic works is £ 8·4 per horse-power installed, and the maximum, £ 79·6 per horse-power. For the majority of the installations the cost lies between £ 25 and £ 45. The cost of the electrical generations switch boards, etc., and transmission lines, also varies greatly, ranging from £ 1·25 to £ 28·4 per h. p., while the cost of the turbine ranges from £ 4 to £ 8 per h. p. The working costs vary between £ 1·3

* British Water Power Committee.

and £ 6·8 per electric h. p. year with an average value of £ 3. From these figures it appears that on the average, making an allowance of 15 per cent. for interest and depreciation, the cost per electric h. p. per annum is, in the neighbourhood, £10·5.

" In many installations, however, the cost is very much less than this. The Ontario Power Co., for example, is able to supply power to the Hydro-Electric Commission of Ontario at £ 1·8 per electric h. p. per annum. It is estimated that many of the large powers in Canada can be developed, a total cost, including all generating machinery and transmission lines, ranging from £ 12 to £ 20 per electric h. p., in which case the cost per h. p. annum should not exceed from £ 2 to £ 3.

" The cost of such power is made up mainly of charges, against capital, interest, depreciation, sinking fund charges, taxes and insurance, which are usually much greater than water charges and cost of operation, maintenance and supplies The capital charges vary widely with the local circumstances and physical characteristics of the site.



FALL IN FEET	Capital cost per kW. in Rs. — Machinery.	Building.	Total.	Working expenses at 11¼ o/o per kW. Year. — Installed in R.	Peak in Rs.	At 50 o/o L.F. in Rs.	H. — Unit in pies.
75	100	60	160	18.00	24.0	30.00	0.65
70							
65							
60	120	65	185	20.81	27.50	34.50	0.76
35							
50	130	70	200	22.50	30.00	37.50	0.82
45							
40	145	75	220	24.75	33.00	41.25	0.90
35							
30	160	80	240	27.00	36.00	45.00	0.99
25	170	80	250	28.5	37.56	46.93	1.03
20	190	95	285	22.68	42.75	54.38	1.17
15	240	110	350	39.38	52.50	65.63	1.44
10	380	300	380	76.50	102.00	127.50	2.79
			1246	140.90	180.50	233.??	5.18

Comparison of Different Hydro-Electric Schemes.

Place.	Year of construction.	Fall in feet.	Cost in lacs of rupees. Machinery.	Building.	Capacity.	Cost of hydraulic development per kW. rupees.	Cost per unit sold in pies.
Renala ...	1922	7	8·89	4·79	1100	124·6	
Jhelum ...	...	395	...	...	...	565	
Jamken ...	1909	26	0·65	2·65	1100	317	2·8
Amritsar ...	1919	97	3·25	.	525	619	
Patiala ...	...	8	.	...	450	900	
Bahadrabad ...	...	19′	...	.	...	303	9
Bhola ..	...	12′-14′		...	750	447	
Palra ...	...	7′-10′	3·93	1·43	600	955	
Sumera ...	...	15′-19′	.	...	1200 K. V. R	364	
High head falls.							
Gokak ...	...	210	...	...	1900		
Tata Power Co, Ltd. ...	...	1601	...	...	87500	550	
The Andhra Valley Power Supply Co., Ltd.	...	1670	...	...	48000	708	
Tata Hydro-Electric Power Supply Co., Ltd.,	...	1694	...	...	48000	566	
Bhatghar ...	...	46-101	..	...	1000 }	368	
Cordite Factory, Nilgiri...	...	650	.	..	1100 }		
Cauvery ...		410-426	229	..	...	550	0·25 to 0·35
Malakand ...		30	.		250 K. V. R.	630	
Mussoorie ...	...	1000	...	...	1775	240	
Simla ...	...	...	...	..	1250	540	5·5
Darjeeling ..	...	275 }	...	...	...	120	
Phurping ...	...	650 }					
Sundarijal ...	...	678					
Mundi with K.V. Ry. ...	...	1800	925	...	22500	3844	
Mundi without K.V. Ry.	..	1800	625	..	...	2777	
Mundi with K V. Ry. ...	..	1800	925	...	48000	1802	
Mundi without K. V. Ry.	...	1800	625	..	...	1302	

Estimates of the Cost of Developing Various Canadian Powers from Reports of Ontario Hydro-Electric Power Commission.

Location of proposed development.	Natural head.	Available head in feet.	Power developed, h. p.	Estimated capital cost.	Cost per h. p.
(1) Healey's Falls, Lower Trent River.	...	60	8,000	$ 675,000	$84·38
Middle Falls, Lower Trent River.	...	30	5,200	475,000	91·37
Rauney's Fall ...	...	35	6,000	425,000	69·67
Rapids above Glen Miller	...	18	3,200	350,000	109·38
Rapids above Trenton ...	...	18	3,200	370,000	115·63
(2) Maitland River ...	...	80 (5)	1,600	325,000	203·12
Saugeen River ...	...	40	1,333	250,000	187·53
Beaver River (Eugenia Falls).	...	420	2,267	291,000	128·28
Severn River (Big Chute)	...	52 (6)	4,000	350,000	87 50
South River ...	...	85	750	115,000	153 33
(3) St. Lawrence River, Iroquois, Ont.	...	12	1,200	179,000	149·16
Mississippi River, High Falls, Ont. A.	...	78 (7)	2,400	195,000	81·25
Mississippi River, High Falls, Ont. B.	...	78	1,100	123,000	181·82
Montreal River, Fountain Falls, Ont.	..	27	2,400	214,000	89 16
(4) Dog Lake, Kaministiquia River.	347	310 (8)	13,676	832,000	61 00
	347	310	6,840	619,700	91·00
Cameron Rapids ...	39	...	16,350	815,000	50·00
	39	...	8,250	600,000	73·00
Slate Falls ...	31	40	3,686	357,600	97 00
	31	40	1,843	260,000	141 00

Third Report; (5) Dam rather expensive. (6) Head works and canal less expensive than ordinary. (7) With storage developed. (8) Including 3,500 feet of head water tunnel.

Limits of Cost of Existing Hydro-Electric Plants in India.

	Per e h. p. installed.		Per kW. installed.	
	Rs.	£ @ 1s 4d	Rs.	£ @ 1s. 4d.
Maximum	1,170	78	1,560	104
Minimum	179	12	240	16
Mean of installations	674	45	900	60
True mean, *i.e.*, total outlay divided by power installed (approx.) ...	350	23·3	468	31·3

Development Costs of Various American Water Power Plants.

Name or location of plant.	Reference.	Head in Feet.	Horse power capacity at turbine shaft.	Cost.	Cost per h. p.	Notes—see page 40.
1. Chicago Drainage Canal, Lockport, Ill.	Elec. World, 1906, Vol. 47, page 398.	28	15,500	$3,500,000	$ 225·80	*d*
2. Columbus, Ga....	Elec. World and Engr., 1904, Vol. 43, page 165.	40	9,000	450,000	50·00	*c* and *e*
3. Catawba, S. C.	Eng. Record, 1904, Vol. 50, pages 114, 129.	25	10,000	1,100,000	110 00	*d* and *f*
4. Tariffville, Conn.	Amer. Electrician, 1900, Vol. 12, page 107.	31	2,300	....	125·00	*d*
5. Delta, Penna ...	Eng. News, 1898, Vol. 39, page 250.	42	550	30,000	54·00	*d* and *g*
6. Lachine, Montreal.	Elec. World, 1898, Vol. 31, page 744.	16	6,600	957,200	145·80	*d* and *h*
7 Winnipeg, Manitoba.	Elec. World, 1906, Vol. 47, page 1291	40	25,600	4,000,000	156·25	*d* and *i*
8 Manchester, N. H.	The Engr., 1902, Vol. 39, page 64	30	6,000	....	66 00	*a* and *j*
9. Lowell, Mass.		13	......	... .	110 00	*a* and *j*
10 Lowell, Mass.		18	......	....	57 00	*a* and *j*
11. Big Cottonwood, Utah		370	3,000	325,000	108·25	*d* and *k*
12 Lawrence, Mass		...	1,000	....	67·50	*a* and *j*
13. Spier Falls, N. Y	Scien. Amer., Sept. 12, 1903.	90	50,000	2,100,000	42 00	*c*

Development Costs of Various Foreign Water Power Plants.

Name or location of plant	Reference.	Head in feet.	Horse power capacity at turbine shaft.	Cost	Cost per h. p.	Notes—see below.
Zürich, Switzerland.	Electricity (N. Y.), 1899, Vol 16. p. 148.	Very low	25,300	$4,650,000	$183·90	*d* and *l*
Rhinefelden, Germany.	Electrician (London), 1897, Vol. 38, p 716	10-16	15,000	1,225,000	81 70	*c*
Paderno, Italy ..	The Engr., 1902, Vol. 39, p. 64	90	13·000	...	120·0	*b*
Champ, France	Eng. Record, 1905, Vol. 52, p. 648	104	6,750	1,000,000	148·00	*d*
Dep't. de l' Isére, France	Elec. Review (Lond.), 1898, Vol. 43, p 475	330	4,000	136,000	34 00	*b*
Dep't. de Jura, France.		65	300	45,000	150·00	*d*
Upper Savoy, France.		450	11,000	182,000	165·50	*c* and *m*
Chevres, Switzerland		14-27	9,600	1,044,000	109·00	*b*
Chèdde, France		455	10,000	...	30·00 42·50	*a* *c* and *n*
Kubel, Switzerland.		296	5,000	1,074,000	215·00	*d* and *o*
Schaffhausen, Germany.		13·8-15·8 11·5-14·8	2,700	365,000	135·00	*d* and *p*
Gersthofen, Germany	Die Ausnutzung der Wasserkräfte, page 198—E. Mattern.	32·8-34·4	6,000	812,500	135·00	*b*
Augsburg, Germany.		...	9,100	1,875,000	206·00	*d*
Heimbach, Germany.		230-360	16,500	2,125,000	130·00	*d* and *q*
Lyon, France		33-40	22,750	6,500,000	287·50	*d* and *r*
Mühlhausen, Germany.		24-30	23,000	3,075,000	132·50	*b*

Notes in Tables on pages 39 and 40.

Estimated Yearly Operating Expenses of Generating Plant from Reports of Ontario Hydro-Electric Power Commission.

Location of plant.	Horse Power.	Net H. P. transformed for transmission.	Operating Expenses including administration.	Maintenance and repairs.	Depreciation.	Interest at 4 °/o	Water rental.	Yearly charge.	Yearly cost of transformed 24 hour power.
1. Niagara plant ...	50,000	48,750	$57,900	$115,700	$86,800	$231,400	$52,000	$544,300	$11'16
	75,000	73,125	70,200	140,400	105,300	280,800	65,000	661,700	9'05
	100,000	97,500	86,300	172,600	129,500	345,200	77,500	811,100	8'32
2. Middle Falls	5,200	4,990	11,875	9,500	9,500	19,000	...	49,875	10'00
Healey's Falls ...	8,000	7,680	16,875	13,500	13,500	27,000	...	70,875	9'10
Two above combined ...	13,200	12,670	23,000	23,000	23,000	46,000	...	115,000	9'08
3. Maitland River ...	1,600	...	5,665	2,751	2,755	13,000	...	24,174	...
Saugeen River ...	1,335		4,840	3,247	3,247	9,984	...	21,318	...
South River ...	750	...	4,100	2,620	2,620	4,534	...	13,874	...
Severn River (Big Chute) ...	4,000	...	17,483	8,571	8,571	14,000	...	48,575	...
Severn and Beaver Rivers combined.	6,267	...	23,713	13,968	14,000	25,640	...	77,289	...
4. St. Lawrence River ...	...	1,200	6,864	5,119	5,118	7,151	...	24,252	...
Mississippi River High Falls		2,400	9,391	3,840	3,841	7,777	...	24,849	...
Do. Do. ...	...	1,100	6,390	2,491	2,491	4,908	...	16,280	...
Montreal River Fountain Falls	...	2,400	9,850	3,903	21,622	8,539	...	43,914	...
5. Dog Lake ...	..	13,675	13,760	16,427	15,927	35,278	...	79,392	...
		6,840	11,296	10,632	10,132	24,787	...	56,847	...
Camoeron Rapids ...	...	16,250	16,375	17,327	16,727	32,561	...	82,990	...
		8,250	14,390	11,478	10,978	24,008	...	60,854	...
Slate Falls ...	...	3,686	6,000	6,634	6,334	14,303	...	33,272	...
		1,843	6,000	3,868	3,669	10,400	...	23,957	...

Estimate of Financial Relations of Various Hydro-Electric Projects in Upper Mississippi Valley.

Project.	Continuous h. p..	Installation h. p.	Head.	Cost b.	Installation Cost. Continuous h p.	Installation Cost. Installed h. p.	Estimated practicable output delivered to customer per year.	Annual Expense c.	Cost per kWh. delivered. * — 50 per cent.	Cost per kWh. delivered. ** — 75 per cent.	Cost per kWh. delivered. *** 100 per cent.	REMARKS.
							kWh.		Cents	Cents	Cents	
1	2,800	8,500	39 29 20	$1,520,000	$543 00	$179·00	16,650,000	$175,500	2·10c	1·40c	1·05	Cost does not include reservoir or flowage.
2	1,500	3,900	61	1,008,000	674 00	259·00	8,500,000	83,000	2·04	1·36	1·02	
3	1,250	3,100	50	869,000	695·00	280·00	6,970,000	72,475	2 18	1·45	1·09	
4	2,650	6,000	108	1,520,000	574·00	253·00	15,070,000	122,175	1·694	1·129	847	
5	625	1,500	17	275,000	440·00	183·00	4,160,000	33,600	1·614	1·076	·807	
6	8,500	12,000	80	1,380,000	162·00	115·00	45,700,000	111,600	·49	·327	·245	No transmission line, power to be used at site.
7	4,300	9,000	40	2,042,800	473·00	227·00	23,300,000	163,700	1·40	·932	70	
8	1,215	2,700	24	247,780	204·00	92·00	5,570,000	25,822	·93	·62	·465	
9	740	2,000	60	139,000	188·00	70·00	3,500,000	18,120	1·04	·693	·52	No flowage.
10	2,200	5,600	100	271,000	123·00	48·00	10,000,000	34,680	·692	·461	·346	No flowage
11	4,000	10,000	17	2,100,000	525·00	210·00	32,000,000	150,000	938	·625	·469	
12	6,000	15,000	25	3,000,000	500·00	200·00	60,000,000	225,000	·750	·50	·375	
13	3,750	7,500	85	1,300,000	350·00	175·00	18,000,000	110,000	1·222	·815	·611	
14	3,100	5,860	17	1,436,100	463 00	245·00	16,650,000	138,000	1·66	1·107	·83	
15	1,200	4,000	110	271,400	226 00	68 00	6,700,000	30,130	·90	·60	·45	Does not include flowage or financing cost.
16	...	8,000	90	3,000,000		375·00	16,000,000	248,000	3·16	2·107	1.58	
17	..	8,000	90	3,362,000	..	457·00	40,000,000	446,000	2·23	1·487	1·115	
18	...	3,000a	30	361,000	...	120·00a	6,300,000	47,450	1·65	1·10	825	With small Steam Plant.
19	3,250	4,500	42	750,000	230 00	166·00	15,000,000	90 000	1·20	·80	·60	

=Approximate.
b=Inc. no value for Water Power Rights.
c=Fixed charges. Operating Exp. Depreciation and Repairs Taxes and Ins.

* Based on Sale of 50 per cent. possible Output.
** Based on Sale of 75 per cent. possible Output.
*** Based on Sale of Entire possible Output.

Comparative Cost of Water Power Plants, in U. S. A.

Capacity	Head.	Cost in dollars of Water Power Development.			
		Without dam.	With dam.	With dam and electrical equipment.	With dam, electrical equipment and transmission line.
8,000	18	63·50	86	115	150
8,000	80	21	39	60	90

Statement showing capital and generating cost of Steam Stations at Lahore or Ludhiana.

Installed capacity in kW.	6,300	9,450	12,600
No. of sets of 3,150 kW. ...	2	3	4
Peak load in kW. ...	3,150	6,300	9,450
Capital cost in lacs of Rupees ...	14·0	21·0	18·0
Capital cost per kW. installed in Rs.	222	222	222
Yearly Working Expenses :—			
Interest on capital at 6 % ...	84,000	1,26,000	1,68,000
Depreciation at 5 % ...	70,000	1,05,000	1,40,000
Establishment ...	48,000	50,000	52,000
Stores for maintenance and repairs.	12,000	18,000	24,000
Total ...	Rs. 2,14,000	Rs. 2,99,000	Rs. 3,84,000
When load factor is 50 % :—			
No. of units generated ...	13,797,000	27,594,000	41,391,000
Coal consumption per kW. in lbs. ...	2·3	2·2	2·1
Cost per unit generated in pies (excluding fuel).	2·98	2·8	1·78
Fuel cost at Rs. 13-12-0 per ton in pies.	2·71	2·60	2·46
Total cost per unit generated in pies	5·69	4·68	4·24
When load factor is 80 % :—			
No. of units generated ...	22,075,700	44,151,400	66,227,100
Coal consumption per kW. in lbs. ...	2·1	2·0	1·9
Cost per unit generated in pies (excluding fuel).	1·86	1·30	1·11
Fuel cost at Rs. 13-12-0 per ton in pies.	2·46	2·36	2·25
Total cost per unit generated in pies	4·32	3·66	3·36

Remarks :—These figures, *i.e.*, 3·36 to 5·69 pies per unit, are based on actual test by Messrs. Siemens—vide pages 193-202, Vo. IV, No. 9, dated 1928, in their review for steam stations having installed capacity 6,300 kWs. costing 14 lacs to 12,600 kWs. And for the capacity of 48,000 kWs, which is the capacity of Mandi Hydro-Electric Scheme, the generating cost will further go down as the coal consumption will be in the neighbourhood of 1·0 lb. per unit for such big plant.

Cost of producing Electric current by the Companies in the United Provinces per unit :—Cauvery 0·25 pie to 0·35 pie ; Calcutta ·48 anna.
Lucknow ·42 anna. Agra ·46 anna. Benares ·333 anna Mussoorie ·09 anna.
Allahabad ·23 anna. Bareilly ·62 anna. Cawnpore ·15 anna. Nainital ·03 anna.

Table showing Average Power Developed and its Cost per h. p. in 22 Steam Power Plants.

OUTPUT.		Operating expenses, per h. p.	Fixed charges, per h. p.	Total Cost, h. p. per annum.	Cost per h. p. hour, etc.
Average h. p. developed.	No. of days per annum.				
12·4	361	$ 147·93	$ 25·40	$ 173·33	5·648
20·9	365	123·12	28·42	151·54	1·868
21·5	361	90·47	17·80	108·27	2·918
32·9	330	22·56	5·83	28·39	·832
36·7	365	137·25	96·70	233·95	2·811
42·4	365	86·38	63·20	149·58	1·701
53	309	56·94	19·51	76·45	1·596
58·8	365	97·30	33·82	131·12	1·613
70·4	365	101·69	20·78	122·45	1·641
129·3	365	30·14	9·41	39·55	·871
166·7	313	15·19	4·47	19·66	·639
173	313	22·66	5·83	28·39	3·333
210·9	290	40·33	7·86	48·19	·693
296·7	297	45·56	7·81	53·37	·749
926	307	11·73	8·77	20·50	·691
1,010·0	306	15·70	7·74	23·44	·794
1,174·8	306	10·19	5·50	15·69	·531
1,278·7	293	10·49	6·23	16·72	·590
1,345·5	365	23·28	9·42	32·70	·820
1,352	365	33·03	29·41	62·44	·713
1,209·7	306	13·40	6·63	20·03	·677
2,422	306	15·67	6·73	22·40	·757

Mr. H. A. Fosterr Transaction Am Just Vol. 14, p. 385.

Cost of Operation of Various Street Railway Power Stations.

Capacity in 1,000 kW.	Type.					Period averaged days	Load factor, per cent.	Labour						Fuel.				General expenses per kW. hour.	Total operating expenses per kW. hour.
	Engines.	Generators.	Belted or D. C. 1.	Simple or compound.	Non-condensing or condensing.			No. of shifts.	Length of shifts, hours.	Total shifts, hours.	No. of men per 1,000 kW.	Rate of pay per hour.	Per kW. hour output.	Lbs. per kW. hour output	Price per net ton	Anthracite or bituminous.	Cost per kW. hour.		
													Cts.				Cts.	Cts.	Cts.
3·6	3	3	D. C.	C.	C.	365	33½	3	8	8,760	1·94	0·27	0·157	2·2	3·00	B	0 33	0·093	0·58
1·4	5	8	B.	S.	N.	365	23	2	12	8,760	3 7	·20	·32	6 7	2·24	A.	·75	·33	1 40
1·0	3	7	B.	C.	C.	365	23	2	12	8,760	3 7	20	·31	4·3	2·93	B.	·63	·11	1·05
2·6	4	4	D. C.	S	N.	365	42	2	12	8,760	5·0	·22	·16	5 6	1·44	B.	·47	·09	·72
1·6	10	16	B.	S. C.	N. C.	183	41	2	12	4,380	6·3	·16	·25	5·6	2·10	B.	·68	·14	1 07
1·6	8	8	B.	C.	C.	365	33	3	8	8,760	3·7	20	·23	5 0	3·30	B.	83	09	1·15
0 8	2	4	B.	C.	C.	365	24	2	12	8,760	3 7	·18	·28	5·0	1 43	B.	·36	06	·70
0·6	3	3	B.	C.	C.	183	16	2	12	4,380	·3	·25	·51	6·0	3·44	B.	1 03	·16	1·70
4·0	10	9	B.	S.	N.	365	33	2	12	8,760	...	...	...	5·6	1 05	...	·29	...	·56
0·8	7	12	B.	S.	N.	365	19	2	12	8,760	...	...	..	6 8	1·15	...	39	...	1·20
1·4	4	8	B.	S	C.	31	23	2	12	744	5·0	·17	·37	7 0	1 63	...	·57	·08	1·02
1 9	5	5	B.	C	C.	365	27	2	10	7,300	3·2	·21	·25	3·9	2·80	...	·55	·23	1 03
1·7	5	5	B.	C.	C.	365	23	2	10	7,300	3·0	·21	·27	3 3	3 00	B.	·49	15	·91
0·9	3	3	B.	C.	C	365	32	2	10	7,300	4·5	·21	·29	4·4	2·84	B.	·62	·19	1·10
5·3	5	30	B.	C.	C.	365	31	2	12	8,760	3 5	·20	·23	3·5	3·00	B.	·52	09	·84
1·5	4	12	Both	C.	C.	365	16	2	12	8,760	5 0	·20	·62	4·3	3 08	B.	·66	22	1·50
2·0	4	4	D. C.	C.	C.	183	57	2	12	4,392	5 8	·20	·18	5·3	1 00	B.	·27	·12	·57
1·2	1	1	D. C.	C.	C.	365	37	2	12	8,700	2·1	·26	15	4·7	1 24	B.	·29	·04	·48
1·5	3	11	B.	S.C	N.	151	11	2	12	3,624	8 0	·16	1·10	7·3	1 75	5A 2	·63	·12	1·85
2·9	6	6	D. C.	C.	N. C.	365	36	3	8	8,760	2·7	23	·17	3 0	1·60	A.	·24	·11	·52
1 8	3	6	B.	C.	C.	365	45	3	8	8,760	5 0	26	·29	5 7	1·60	A.	45	·18	·92
5·3	5	10	Both	C.	C.	365	15	2	12	8,760	3·7	·16	·29	3·7	2 70	6B.3	·50	·14	1·03
6 0	6	12	B.	C.	C.	365	30	3	8	8,760	3·3	24	·26	3 5	1·60	A.	·28	·15	·69
1·0-	4	7	Both	C.	C.	30	28	2	10	600	3 0	·15	·16	4·7	1·90	B.	45	·05	·66
0·75	4	4	D C.	C.	C.	183	20	2	12	4,380	8·0	·12	·41	5·1	2·12	·5A	·54	·21	1·16
0·6	2	2	D. C	C.	C	365	35	2	10	7,300	6·7	·15	29	3·3	2·75	B.	·45	·28	1·02

Working Expenses.—The cost item "wages of workman" covers wages at generating station. The wages incurred on the supervision and maintenance of the distributing system are included in "repairs and maintenance."

Average price obtained per unit.—"Private supply" = Revenue received from private consumers for light. power aud heat.

"Public Lighting." = Revenue from Street Lighting after deducting the outlay on trimming. cleaning and maintaining the public lamps.

Table of Electric Supply Costs and Records

PLACE.		CAPITAL.	REVENUE.		COSTS.
Local Authorities.	Prop. of area of supply in thousands.	Total expenditure at end of year.	From Electricity supply.	From meter rents and other sources	Working costs.
Metropolitan.		£	£	£	£
Battersea ...	169·0	1,155,187	186,379	3,351	92,554
Hammersmith ...	136·3	1,090,036	230,989	1,568	134,735
Southwark ...	115·0	242,051	67,132	1,286	45,255
Aldershot ...	25·0	102,490	23,126	1,422	15,159
Bexley ..	35·0	163,052	29,044	1,654	16,602
Bingley .	20·0	85,340	22,819	2,752	17,498
Birmingham ...	1030·0	11,033,620	1,816,876	5,358	937,910
Bo'ness (May) ...	10·2	74,832	14,090	421	8,109
Cannock ...	50·0	153,948	27,528	3,038	121,520
Coventry ...	173·2	1,871,002	342,792	1,753	195,214
Croydon ...	216·6	1,326,993	300,659	11,431	28,700
Derby ...	147·7	1,669,744	204,994	438	90,349
Dundee (May) ...	168·3	1,293,572	235,480	*Nil*	27,708
Gravesend ...	48·0	449,694	73,014	3,316	137,898
Haslingden ...	17·5	85,074	21,351	708	15,268
Huddersfield (Dec.)	133·3	786,848	215,863	2,736	114,908
Hull ...	339·9	2,600,695	394,165	12,946	203,626
Keighley ...	55·7	555,670	83,973	3,853	45,240
Lincoln ...	66·1	608,801	96,865	581	48,770
Liverpool ...	1016·6	7,973,396	1,325,440	39,960	697,925
Londonderry ...	45·2	342,610	40,721	1,995	15,541
Luton ...	115·4	717,977	153,449	3,225	91,079
Maidstone ...	40·0	559,398	97,697	4,569	57,246
Manchester ...	855·0	10,895,803	1,739,982	32,130	1,077,626
Middleton ..	31·0	132,544	43,752	1,468	30,976
Motherwl. & W. (May).	64·8	205,768	57,144	*Nil*	41,000
Norwich ...	190·4	1,399,840	224,851	3,709	121,813
Oban (May) ...	6·5	48,278	8,898	122	4,683
Peterborough ...	60·0	838,456	121,067	1,785	67,712
Rotherham ...	105·4	1,601,354	298,468	2,324	148,853
Rughby ...	25·2	54,897	26,278	1,793	16,720
Sheffield ..	515·4	5,379,410	872,832	18,178	440,654
Stockport ...	127·8	1,141,797	199,603	997	114,944
Stoke-on-Trent ...	309·8	1,669,831	215,559	4,777	101,788
Stretford ...	67·9	850,959	219,384	2,713	150,673
Wallasey ...	101·3	781,444	149,717	2,738	87,920
West Ham ...	320·0	2,017,638	392,607	3,157	230,690
Winchester ...	24·8	196,758	33,721	2,250	19,439
Yarmouth ...	91·0	692,672	95,934	5,485	36,985
York ...	84·0	808,111	126,154	4,602	56,966

(Year ended March, 1930).

Gross Profit.		Interest & Special.	Loan Repayment.		Financial Result.	Output
Before Providing for Interest and Loan Repay.	Per cent. to Capital (outlay during year averaged)	Interest and Special Expenditure, less Crs.	Amount Provided for Loan Repayment.	Per cent to Capital (outlay during year averaged).	+ — Surplus — — Deficit	Number of Units Sold.
£	%	£	£	%	£	Units
97,176	8·91	32,652	41,280	3·79	23,244 +	30,569,348
97,822	9·19	33,894	41,587	3·91	22,341 +	37,232,694
23,163	9·66	5,057	7,604	3·17	10,502 +	5,991,272
9,389	9·77	3,610	3,288	3·42	2,491 +	1,354,021
14,096	9·23	4,617	4,838	3·17	4,641 +	3,345,353
8,073	9·94	4,020	4,137	5·09	84 —	2,523,223
884,324	8·48	408,460	300,325	2·88	175,539 +	357,820,996
6,402	8·56	1,866	3,542	4·73	994 +	1,508,357
9,046	6·27	7,415	3,423	2·37	1,792 —	5,001,929
149,331	8·34	66,877	63,123	3·53	19,331 +	91,694,550
183,390	14·41	123,347	28,027	2·20	32,016 +	42,091,007
115,083	7·28	52,020	41,626	2·63	21,437 +	33,953,695
107,772	8·49	53,523	53,835	4·24	414 +	46,056,574
38,432	8·86	12,591	11,848	2·73	13,993 +	16,342,040
6,791	8·60	2,743	2,490	3·15	1,558 +	3,468,971
103,691	13·45	51,278	19,087	2·47	33,326 +	41,538,456
203,485	8·47	88,620	59,980	2·50	54,885 +	84,687,880
42,586	7·85	23,163	17,822	3·28	1,601 +	14,635,296
48,676	8·09	22,529	25,992	4·32	155 +	20,133,128
667,475	8·64	353,691	193,446	2·50	120,338 +	265,083,853
27,175	8·02	8,976	14,730	4·35	3,469 +	3,537,287
65,595	9·84	23,391	25,664	3·85	16,540 +	40,014,410
45,020	8·56	17,106	21,318	4·05	6,596 +	27,552,834
94,486	6·50	375,033	273,409	2·56	46,044 +	392,983,029
14,262	11·06	8,567	3,448	2·67	2,247 +	10,324,492
16,144	7·87	6,710	8,937	4·36	497 +	9,790,933
106,747	8·17	49,277	33,024	2·53	24,446 +	33,140,302
4,337	9·00	1,678	[illegible],739	3·61	920 +	.530,093
55,140	6·77	33,740	30,213	3·71	8,813 +	28,340,000
151,939	9·70	61,692	49,577	3·16	40,670 +	112,638,787
11,351	21·46	8,001	118	·22	3,232 +	2,521,299
450,356	8·61	188,697	174,715	3·34	86,944 +	223,934,191
85,656	8·00	33,706	42,11[illegible]	3·93	9,838 +	44,924,111
118,048	7·46	57,366	52,935	3·34	7,747 +	38,245,303
71,424	9·43	41,438	25,689	3·39	4,297 +	67,766,495
64,535	8·59	39,868	22,020	2·93	2,647 +	47,102,430
165,074	9·05	82,489	54,706	3·00	27,879 +	98,074,425
16,532	8·67	8,474	6,346	3·33	1,712 +	2,531,551
64,434	9·62	28,181	29,245	4·37	7,008 +	12,503,558
73,790	9·86	24,735	19,517	2·61	29,538 +	27,746,651

Table of Electric Supply Costs and Records

PLACE.	WORKING COST PER UNIT SOLD.						
Local Authorities.	Coal and other fuel.	Oil, Waster, Water and Stores.	Wages of Workmen.	Repairs and Maintenance	Rent, Rates, and Taxes.	Management Salaries, Office and Legal Exs, Insce &c.	Total Working Costs.
Metropolitan.	d.	d.	d	d	d	d	d.
Battersea ...	·25	...	·09	·12	·10	·17	·73
Hammersmith ...	·34	·01	·11	·20	·07	·12	·85
Southwark ..	·57	·05	·16	·42	·30	·31	1·81
Aldershot ...	(Engy.	b't 1·33	Distrib.	·71)	·27	·38	2·69
Bexley ...	(Engy.	b't ·61	Distrib.	·25)	·13	·20	1·19
Bingley ...	(,,	b't ·89	Distrib	·44)	·14	·19	1·66
Birmingham ...	·19	·01	04	·19	·13	·07	·63
Bo'ness (May) ...	(Engy.	b't ·69	Distrib	·18)	·25	·17	1·29
Cannock ...	(Engy.	b't ·69	Distrib	·12)	·06	·16	1·03
Coventry ...	·24	...	·04	·07	·08	·08	·51
Croydon ...	·24	...	·06	·19	·11	13	·73
Derby ...	·13	·01	·08	·16	·12	·14	·64
Dundee (May) ..	·17	..	·04	·23	·14	·08	·66
Gravesend ...	·27	...	·06	·08	·09	·06	·56
Haslingden ...	(Engy.	b't ·64	Distrib.	·20)	·09	·13	1·06
Huddersfield (Dec.	·21	.01	·06	·19	·14	·05	·66
Hull ...	·18	...	·08	·15	·07	·10	·58
Keighley ...	·36	...	10	·10	·08	·10	·74
Lincoln ...	·17	...	·07	·10	·11	·13	·58
Liverpool ...	·19	·01	·07	·10	·18	·08	·63
Londonderry ...	·42	·01	·31	·12	·03	·16	1.05
Luton ...	·35	...	·05	·07	·03	·05	·55
Maidstone ...	·19	...	·05	·12	·06	·08	·50
Manchester ...	·15	·10	·11	22	·10	·07	·66
Middleton ...	(Engy.	b't ·49	distrib	·11)	·06	·06	·72
Motherwl&W(May	(Engy.	b't 55	distrib.	·27)	·12	·06	1·00
Norwich ...	·26	·01	·06	·19	·15	·21	·88
Oban (May) ...	·39	·07	·28	·92	·05	·41	2·12
Peterborough ...	·22	·01	·11	·13	·04	·06	·57
Rotherham ...	·14	...	·03	·07	·04	·04	·32
Rughby ...	(Engy	b't 1·13	Distrib.	·07)	·13	·26	1·59
Sheffield ...	·20	...	·04	11	·07	05	·47
Stockport ...	·19	...	·06	·17	11	·08	·61
Stoke-on-Trent ...	·17	·02	·07	·16	11	·11	·64
Stretford ...	·26	·01	·04	14	04	·04	·53
Wallasey ...	·16	·01	·04	13	05	·06	·45
West Ham ...	·22	..	·06	·13	·09	·06	·56
Winchester ...	·39	...	·25	·43	·30	·47	1·84
Yarmouth ...	·23	·01	·08	·13	·11	·15	·71
York ...	·16	...	·08	·11	·07	·07	·49

Year ended March, 1930).

Supply Records.					Supply Records.			
Average Price Obtained.								
Private Supply.	Public Lighting.	Traction Supply.	Bulk Supply.	Total Supply.	Units sold for Private supply per head of Population.	Total Connections.	Maximum Load on Generators and Bulk Supply Purchased.	Undertaking Load Factor Units × 100. / Max L × Hrs.
d.	d.	d	d.	d.	Units.	Kw	Kw	%
1·54	1·10	...	·33	1·46	162·3	35,885	13,500	32·66
1·55	·67	...	1·04	1·49	245·3	31,500	16,100	32·32
2·75	1·84	...	—	2·69	48·5	6,734	4,418	18·84
4·10	4·00	...	—	4·10	51·5	2,489	997	20·24
2·18	3·18	1·37	4·19	2·08	74·0	8,125	1,606	26·92
2·26	1·97	1·73	—	2·17	104·8	6,828	1,347	25·14
1·24	1·51	1·07c	·65	1·22	300·8	332,928	155,700	31·62
2·21	3·02	—	—	2·24	143·0	2,726	595	33·22
1·25	7·09	—	—	1·32	98·9	6,095	1,620	39·81
·90	·99	·58	—	·90	520·4	72,533	38,070	31·98
1·86	2·16	1·04b	·24	1.71	152·9	67,148	19,500	31·36
1·52	1·40	1·20b	·29	1·45	196·7	60,019	16,892	29·64
1·25	1·31	·97c	—	1·23	244·7	78,032	23,700	25·80
1·06	1·42	—	—	1·07	332·8	11,749	6,100	36·89
1·53	1·00	1·00	—	1·48	178·9	8,040	1,521	27·86
1·31	·46	1·01c	—	1·25	248·5	65,264	17,450	33·09
1·12	1·08	1·73	1·31	1·12	247·9	—	32,800	35·79
1·59	·92	1·11b	79	1·38	185·9	15,697	7,732	27·36
1·15	1·98	—	—	1·15	303·8	24,606	8,600	33·30
1·27	1·34	1·01c	·61	1·20	196·4	177,991	102,490	35·72
2·98	1·64	—	—	2·76	70·9	6,461	2,150	23·70
·96	1·40	·80b	·43	·92	315·6	51,876	14,630	39·60
·85	·88	·83b	—	·85	664·7	21,750	9,040	43·62
1·25	1·54	·86c	·47	1·06	307·1	424,613	159,710	35·43
·99	1·54	1.11b	—	1·02	295·0	9,016	4,733	27·90
1·43	·94	1.50	—	1·40	135·6	17,171	3,682	33·05
1·72	·80	71	1·42	1·63	157·1	57,052	16,700	28·12
4·10	3·17	—	—	4·03	. 4·8	1,185	269	25·02
1·02	·88	1·58b	1·25	1·02	152·4	23,256	9,250	40·23
·71	—	1·09b	·38	·63	753·0	81,542	35,500	40·08
2·50	—	—	—	2·50	100·0	5,383	1,271	24·38
·96	·92	·80c	·50	·93	369·6	290,606	78,987	38·90
1·19	·56	·72c	·54	1·07	275·9	42,532	24,130	26·22
1·38	1·17	—	·67	1·35	116·0	38,185	15,400	34·31
·76	·80	1·17d	—	·78	948·0	47,712	21,110	39·80
1·34	·50	1·03b	·40	·76	170·3	34,269	15,050	39·47
1·14	·56	·87b	·55	·96	192·6	61,193	36,500	36·16
3·26	1·75	—	—	3·20	97·7	7,267	1,473	22·97
2.58	1·18	·94c	·88	1·84	75·9	24,757	6,472	25·68
1·10	1·53	·76c	—	1·09	301·1	—	10,960	35·97

* "Necessity for Preliminary Investigation

"In spite of the great importance of water-power, many of the potential powers in existence must of necessity prove economically useless, either on account of their great distance from centres of industry, the lack of transport facilities, or from the fact that the storage necessary to give a continuous or fairly continuous supply would be too costly. Of many potential powers it can be said, without further investigation, that for the present this is, and for a long period to come will be, the case. Of others the reverse is true, and it is evident that the scheme will amply repay development. But in the majority of cases the extent to which a scheme is capable of economic development can only be determined after a careful examination of the catchment area and of the site of the proposed works, after a careful and prolonged investigation of the rainfall and run-off records; and, especially in an undeveloped country, after an investigation of the mineral and forestral or agricultural possibilities of the surrounding region.

"It has usually been understood that the usefulness of a water-supply depends on the possibility of maintaining its uniformity over the whole period of the year, and that the maximum useful power is strictly limited by the minimum power, which, by the aid of any suggested storage system, will be available towards the end of the longest probable period of drought.

"Where the power is utilized for the supply of some industrial centre, this is undoubtedly true; but if the idea were to be generally adopted, it would cut out an enormous aggregate of potential power, more particularly in tropical and semi-tropical countries. The possibility of utilizing flood supplies for seasonal operations in connection with mining, agriculture and forestry, or for the production of nitrates in such cases, would appear to be worthy of close consideration.

"In any case, the possibilities of a given scheme can only be determined after a prolonged hydrographical and meteorological investigation of the site and surroundings. To be of real value such an investigation should extend over a long series of years. Rainfall records, though forming the basis of any such investigation, are only of partial assistance in dealing with water-power questions. The actual run-off from the catchment area is the all-important factor, and the ratio of run-off to rainfall varies with the physical characteristics of the area, the vegetation, and the climate, so that rainfall gaugings cannot be substituted for the more laborious and costly frequent gaugings of flow. It must be emphasized that each scheme of development requires

* Adopted from the Report of the British Water Power Committee.

independent investigations to determine completely the local conditions governing the flow from the area intended to be utilized.

"Much can be done to ascertain the approximate possibilities of a potential scheme before deciding to incur the heavy cost of a detailed survey by—

(1) Installation and continuous recording of river gauges on all likely channels.

(2) Installation and recording of rainfall gauges at suitable places.

(3) Observation of river discharges for a series of gauge readings.

"If a reasonably long record of rainfall exists, the determination of the run-off for a few years will serve to give a relation between precipitation and run-off which can be carried back as far as the rainfall records go. The initiation of operations (1) and (3) costs little, and no time is lost in collecting the more important data.

"While this is true, it should be borne in mind—

(1) That to be of reliable value from a commercial point of view the hydrometric studies must give a continuous record for a number of years, and show not only the minimum low water flow, but also the maximum flood conditions that have to be met in designing the headworks.

(2) That the investigation of suitable rivers should include contour plans of the sites, profiles along the entire power reach of the river, and along the banks; also studies of lakes or lochs for storage, where they exist, and of the possibility of interconnecting two or more such lakes to feed one large project. These studies should be in sufficient detail to allow of preparing preliminary plans and estimating capital and operating costs, in order to demonstrate the capacity available and the commercial feasibility of development.

(3) That to develop the most obvious power site on a river without full investigation of the whole power reach of the river may not secure, and may make it impossible to secure, the maximum advantageous use of the river by the development to two or more sites.

(4) That to secure the maximum possible use of a river the investigations should therefore be made by the Government rather than by private interests.

(5) Especially is this the case where storage may be developed, in order that the maximum possible storage may be secured, and that the water may be equitably distributed to, and the

cost of the works equally borne by, the various interests benefited. Proper storage may greatly improve flood conditions and enhance the value of land as well as increase the power available.

(6) That, without complete surveys the capacity of a river cannot be accurately judged. The pondage created by the dam will in many cases more than take care of the daily peak load, thus increasing the power available beyond that due to the minimum low water flow, and this may be still further increased by storage at the head waters. The power capacity of a river may sometimes be increased by such means by 100 or 150 per cent., or more."

Procedure

For undertaking a hydro-electric scheme, consider the sources of data available for the investigation and report. —(1) First procure a map and existing reports of the place where the fall is situated, if they are available. "For general geographical purposes the modern atlas sheets of the survey of India," "India and the adjacent countries" on the scale of one millionth are admirable. The "Political Edition" shows the boundaries of the states, provinces, etc., while the "Layered Edition" shows the nature of the country in extraordinary relief. For the initial examination of a possible water-power site no map of a scale of less than 1 inch to a mile is of much value, and then only if it is contoured, details of personal investigation report should be indicated.

Thus preliminary to your starting for reconnaissance work, determine your scope of investigation, and study carefully the largest contoured maps which are available, and after reconnaissance submit as comprehensive a report as circumstances permit. *

Reconnaissance

(1) Description including location of the site of the river and the tributary of the river reached from via (route, rest-house, etc.) by motor or other vehicle or on foot, and transport available.

(2) Type of project—whether high, medium or low heads, classification of water-power and hydraulic lay-out. Mark off which division or sub-division appears most likely from inspection of site. See chapter III.

(3) Water resources of the scheme.

Head or Fall.—Approximate gross height of natural waterfall in feet. State your method of determination of the same.

* Reconnaissance of site as suggested in the Hydro-Electric Survey of India, Vol. III, Triennial Report, by J. W. Meares, pp. 31-35. And the Scheme developed by the Dominion Water Power Branch, Department of Interior Canada, 194 Annual Report of the Department.

Approximate gross height of artificial fall or of natural fall increased by extra head obtainable above or below same, by means of an open channel, in feet.

State method of determination by level aneroid or map contours. The gross head should be estimated from the power house site to the forebay site in the case of high and medium heads. For falls classified as "low" take the vertical height between head waters and tail waters—

Amount of power available.

(*a*) with storage,

(*b*) without storage.

In each case the maximum, mean and minimum values are to be given.

(*a*) The head or fall—State your method of determination of the same.

(*b*) Storage possibilities—

(*i*) Location of possible requisite in sites.

(*c*) Flow and storage -Whether it is possible to mature the project—

(*i*) On the natural flow alone with or without regulating storage only as in low falls.

(*ii*) On natural flow supplemented by storage in dry months as in medium or high falls.

(*iii*) On storage alone as in high falls only.

(*d*) Whether storage is readily available.

(*e*) Flow of rivers—If known, give the maximum recorded flood discharge in cusecs.

If known, give the maximum, mean and minimum recorded discharge cusecs for each month ; also minimum for the year.

Recorded at..........................(location) ; date.

Where records are kept, give the rough average discharge that is found during the dry months in good and bad years : cusecs at....................(location).

Where no regular records exist, give results of any special gaugings available in form :

Year.......................date.....................previous............rainfall of month...........inch, discharge observed..............cusecs. When is the discharge likely to be at its lowest ?

In the case of low head river developments, state whether it is likely that in flood times the tail waters below will back up more than the head waters, and thus diminish the available head. In a gorge, or where there is a gorge near-by down stream, this is likely to be the case. With a river narrow at the headworks

and wide, with a fair bed slope, below the station site, the head will probably be increased.

Is the river liable to sudden spates of great intensity ?

Is it snow-fed from permanent or from winter snow ?

Rainfall ; **catchment** and its geology and topography, precipitation and run-off.

Temperature records are useful as enabling some approximation to the probable evaporation to be made failing any direct measurement.

Catchment area above dam or headworks.

Rain-gauge stations within or near same.

(*a*) Average total rainfall in dry months from..................to ....inclusive...........inches.

(*b*) Minimum rainfall in dry months from..........................to ....inclusive...........inches.

(*c*) Average for rest of year.

(*d*) Minimum for rest of year.

Maximum rainfall in one day, if known ; (small catchments only). Probable annual run-off (known or estimated) in million cubic feet in average year.

Probable annual run-off (known or estimated) in minimum year. Evaporation records, if available, should be considered in this connection.

Regulating Storage.—In the case of high or medium head schemes there should be at least a few hours' supply stored, preferably in or close to the forebay ; but if no site exists, then further back. State if such a site exists and in what position relative to the probable lay-out ; its capacity very roughly, if an intelligent guess will give it ; whether it will be formed by a dam in the main or a subsidiary stream or by an artificial tank ; and any further information as to probable cost of construction, nature of ground, etc., so far as inspection only makes this possible.

Site for regulating storage at distance from forebay site capacity roughly...........cu. ft.

Whether formed by dam in main stream.

do. do. dam in subsidiary valley.

do. do. artificial tank.

Further details from inspection.

Main Storage.—In the case of medium or high fall projects dependent partly or entirely on storage, give (as near as may be) location of dam and nature of ground, very roughly the length and height, with corresponding capacity, so far as inspection or 1-inch map will give any information.

Location of dam (on 1-inch map by square as explained above).

Foundation of dam.
Approximate length of dam
Nature of bottom of ground.
Height likely to be adopted (so far as can be seen without survey).
Probable method of surplussing flood waters.
Probable capacity in million cubic feet (or approximate length and breadth of lake surface).

Statement as to (*a*) effect of eliminating, by cutting across, etc., long tortuous reaches, bends and so forth, showing financial advantages, etc.; (*b*) concrete lining of certain bad reaches where it is known loss of water occurs or where the coefficient of roughness is excessive; (*c*) approximate velocity of flow near and at the site of diversion for low water and for flood water levels ; (*d*) nature of stream bed and banks along the reach, but more particularly for a short distance above and at the diversion site.

If data under paragraph "rainfall ; catchment ; run-off" are scanty, state if there is a reasonable probability of filling the reservoir proposed and whether the conditions render a large carry-over necessary for probable bad years (two fairly bad years are generally more dangerous than one exceptionally bad one between good years).

State nature of ground which will be submerged—whether forest, jungle, waste land, good or bad agricultural land.

Does it contain any villages, temples, etc.

Prior rights above or below site, fishing rights, potable supplies, compensation, ice conditions during winter months, and diversion.

" Headworks

Give particulars, so far as inspection discloses them, as to how the supply from the river (in cases of river development) will be regulated at the headworks, and brought into open channel or forebay, as the case may be.

Probable location and accessibility of headworks.

Notes on the geologic and topographic features.

State if the water is fairly free from, or particularly charged with, silt and detritus in the rains.

State if river is used for timber flotation, and if this is likely to interfere with its development for power.

Type of headworks and diversion."

" Open Channels or Canals

Where the site involves these, the ground should be examined. In the case of low heads, a canal in earth or rock alone will serve,

With medium (or even high) heads, where the power will be large, the same remark applies. In smaller high or medium head projects an artificial duct will generally be indicated : timber seldom lasts well, so masonry or concrete is generally used. Masonry requires special case where crabs are found, as they rapidly cause heavy leakage. For small schemes on high heads even galvanized iron troughing has been used, with a comparatively high slope and amount of load for which each pipe line is intended, and most important details of pipe line, its foundation, backing, anchorage, etc.

State type of channel probable.

State if the ground is favourable or whether special difficulties will be involved.

State if any tunnelling, bridging or syphoning will apparently be necessary, with such brief particulars as inspection discloses.

State approximate length of channel from headworks to forebay. In most cases it is unnecessary at this stage to go into details as to capacity, slope, etc.

Which bank does examination show to be most favourable for the open channel or canal ?"

" Forebay

State position of forebay, suitability of site as regards economy and capacity and whether the site is good. (This may be, but is not necessarily, the means for regulating storage; and probable capacity for pondage and regulating purposes, but it must be large enough to take charge of ordinary fluctuations of load if the regulating storage is at a distance). There must be the possibility of surplussing water not required for the power house and in some cases of dissipating considerable energy in the process. In other cases this is necessary at the main or a subsidiary reservoir. Accessibility and nature of land submerged, type of construction proposed and relative amount and cost of rock-cut excavation and and fill. Its location with respect to the pipe line, etc."

" **Pipe Line.**—Give approximate length of pipe line from forebay to power station site, (alternatives may be suggested, with head in each case). Pipe details need not be gone into, but the line should be as short as practicable.

Statement giving accessibility and suitability of hill side; number of lines and type of pipe or pipes proposed, approximate length of different pipe sections; their thickness and respective diameters, total length of pipe line.

"Power House

Give information as to area of site available; approximate height above river-bed and above stream water-level at low flood level if ascertainable. Suitability as to nature of ground and its extent.

Accessibility of site to good roads and rail roads or navigable river. Type of structure, type and capacity of plant, character and electric equipment and auxiliaries.

As noted in paragraph "Head or Fall," the gross head should usually be estimated from this site.

Probable location.

Area of site approximately.

Height above bed level.

ditto flood level.

Nature of ground for building.

Note that in low head installations and up to about 300 ft. the power house can be placed (if necessary) up to 10 to 24 ft. above low water-level, by the use of draft tubes. Where the height is greater, the turbines can be placed in a pit and connected to the generators by a long vertical shaft.

Will any difficulty occur over the tail race discharge?

Statement dealing with tail race giving (1) probable restricted discharge area in a state of nature, (2) restricted discharge area anticipated by the location of hydraulic works. Length and type of tail race from power house to the stream, showing approximate amount of rock-cut and excavation.

Irrigation Considerations.—Will development of the site in any way interfere with the function of existing irrigation works?

Are there any vested rights such as privately owned water courses for irrigation, or to feed mill wheels, between the proposal headworks and power station site?

Is the district one in which irrigation exists or is needed?

If this is so, would the fall it is proposed to utilize or develop have to be curtailed in order to utilize the tail waters? This will generally not be so if the power station site is at considerable altitude above the neighbouring plains, so that the tail waters would in any case continue their course in the natural channel: on the other hand, there are cases where the tail waters could conveniently be led hereafter directly into a canal, where some power head might have to be sacrificed to give command.

Facilities and Materials.—Will existing roads (rivers) carry the machinery, etc., required?

If so, by what means?

If not, what length of road approximately will be required to complete connection ?

Will there be exceptional difficulties in making this road ?

Are bridges involved ?

Are stone, bricks, lime, suitable sand, kankar, etc., known to be available in the neighbourhood of the works ?

Is labour available or would it have to be obtained from elsewhere ?

Local Rates.—State rates, if known, or probable rates, for :—

Masonry in lime per cent.

Masonry in cement per cent.

Concrete in lime per cent.

Concrete in cement per cent.

Excavation in earth per thousand.

Excavation in hard ground per thousand.

Blasting rock per thousand.

Contour roads, 4 feet, per 1000 r. feet.

Additional information acquired during reconnaissance.

NOTE :—Plans and photographs of headworks site, power station site or any other feature may be of value. A sketch of the plan and longitudinal section and all available run-off, rainfall and gauge records should invariably accompany the report.

Signature.

Designation."

"Survey of Sites

The report should contain maps and photographs of the catchment, reservoir, dam, headworks, conduit, forebay, pipe-line, power house, etc.

When reconnaissance shows that a site is probably worth developing, the data required must be checked by actual survey sufficiently accurately to enable estimates to be framed for the civil engineering works. In the case of canal falls the survey will already exist, and what is required is a detailed report as to the best method of utilizing the fall or falls with rough estimates of the works required. In most other cases there will exist at the best a 4-inch contoured forest map and at the worst an un-contoured (and practically useless) old-style 1-inch map, so that very little exact knowledge can be obtained.

I. Longitudinal Section.—In the first place a longitudinal section should be made of the bed of the river between the limits that may prove useful.

The entire conduit from the intake to forebay, entire pressure pipe line from the forebay to the power house and the profile

of the river or stream above and below the power reach indicating estimated height of backwater during floods.

II. Cross-sections and Contoured Plan:—The next point is to determine on which bank the works may best be placed, and for this purpose, unless the maps give a clear answer, it is necessary to take cross-sections across the valley at intervals, extending up to the highest point where there may be works. This will enable a large scale contoured plan (8 or 16 inches to the mile) to be made, on which the lay-out can be sketched out roughly. The best site for the headworks, the alignment of the canal (if any), the position of the forebay and of the regulating reservoir (if any), the best alignment for the pipes and the position of the power station can be approximately determined on this plan. The sections of the nature of the ground for the foundation of the dam, forebay, regulating reservoir, conduit, surge tank, pipe line, power house, tail race, etc., must be given. There will often be many alternatives to choose from, so that it will be a case of balancing probable extra cost against extra power obtainable on a higher head. It may be noted that if the best site for the headworks is considerably above the highest level on which an open channel appears feasible, the extra head can be sacrificed. Until the survey is complete it is obviously impossible to say exactly what the best lay-out will be.

III. Site of Power House and Forebay and Location of Pipe Line.—A plan and sections on a large scale should next be prepared of the forebay site; of the regulating reservoir if near to or combined with the forebay (none may be needed); of the power house site and tail race. From this the best locations can be definitely determined, and the capacity and rough dimensions of the forebay and regulating reservoir worked out. Pipe design need not be entered upon, and the power house can be left over so long as the plan shows the area available for it. The tail race back to the nearest stream should be dealt with and also the overflow from the reservoir.

IV. Survey of Open Channel.—The route already determined on for the open channel (when one is involved) from the forebay up to the neighbourhood of the headworks will next be surveyed, and its nature, dimensions and slope determined according to the flow to be allowed for. If the regulating reservoir has to be placed somewhere along this channel, instead of close to (or combined with) the forebay, this will involve a break of levels, *i.e.*, it will be fed at top level and will discharge at bottom level. Silt traps may have to be provided for in the run of the channel, unless clear water can be counted on and landslips are improbable. Tunnelling

may be required, as well as means of crossing canals on the route and of allowing overflow at safe points if the channel gets clogged.

V. Headworks.—Similarly the headworks site will be surveyed (it is assumed to have been definitely located before the channel is surveyed) and examined with a view to such a design as will secure the supply in times of low water and both the supply and security in times of flood. Protective works may be necessary to ensure this, probably in the form of boulder crates, and this matter should be examined.

VI. Storage.—If storage on a large scale is involved, the dam site and submerged area will be surveyed so as to determine the dimensions and consequent capacity at draw-off level, and the practicability of the site for construction. The problem of surplussing extra water must also be dealt with in the design by whichever method appears to suit the conditions best.

VII. Project Estimates.—Rough project estimates should then be framed of the quantities involved and approximate cost, particularly of power developed, (*b*) cost of storage, care being taken to indicate very clearly what is included and excluded. It is not intended that the estimates should do more than *indicate whether the scheme will prove cheap, moderately cheap, moderately expensive or very expensive* in terms of rupees per horse power capable of development. Beyond this point the matter can be left over until it is decided to proceed with the works.

VIII. Market for Power.

(*a*) Present.

(*b*) Future, including any obvious possibilities of the development of local industries and of economical power transmission to more remote industrial localities.

(*c*) Length of any transmission line.

IX. When more than one site is investigated, the relative advantages should be discussed.

Note that the *preliminary report* should make it clear whether the scheme is reasonably feasible from an engineering and financial stand-point.

The *final report* should give sufficiently detailed data to enable any financial body definitely to determine the ultimate prospects of the scheme."

"General Report on Site

Such information as is available should be summarized in this form, which is drawn up to be progressive from the worst to the best sites.

I. The site is certainly (very probably) useless as shown by the particulars in Part A-B paragraph above; or because the cost of works especially (the ......) would be prohibitive ; or,

II. It is useless to investigate the site further until gaugings have shown what the minimum flow and/or the yield of the catchment will be; but if these prove favourable, development under a head of about.........feet may be feasible.

III. The site appears to be worth further investigation, but its value for power is very indefinite owing to lack of knowledge of the minimum flow and/or yield of the catchment and for storage possibilities; or,

It is, however, probable that the power capable of development under a head of about.........feet is of the order of :

e. h. p. for the 3-4-5 driest months :
e. h. p. for the rest of the year.

IV. The site is worth a preliminary survey as it appears favourable and can be developed under a head of about.........ft.

It is likely to yield power of the order of :

e. h. p. for the 3-4-5 driest months :
e. h. p. for the rest of the year.

V. The site is certainly worth detailed survey. Under an available head of about..................ft. it will, if the results of the survey are satisfactory, yield not less than—

e. h. p. for the 3-4-5 dry months :
e. h. p. for the rest of the year.

Signature.
Designation."

Resume

Briefly, an economical water-power development depends upon the available *flow*, the *head*, the *cost*, and the *power market*. Hence, at least three of the principal conditions to be determined as accurately as possible to bring about the desired economic results are, apart from cost—(1) a reliable hydrometric survey and analysis of the conditions of the stream and storage under investigation; (2) reliable knowledge of the net static head; and (3) reliable estimate of the load factor and character of the load curves. The commercial value of any water-power would also depend very much upon the size of, and in general the shape of the power market available for selling energy, the proximity of this market, and also on the initial cost of the total development. To market the output from most water-power developments (which are usually situated in a remote part of the country), certain favourable rates must be offered which will interest several different classes of possible *bulk power* consumers.

The **classes of consumers** are broadly as follows :—

(1) Those who intend to have their own steam or oil engine plants but being within commercial distance may subscribe power from the hydro-electric scheme if they are within commercial distance, *e.g.*, flour mills with small engines.

(2) Those who are now already purchasing power from competition undertakings.

(3) Those who are already operating isolated plants of almost any size.

(4) Bulk purchasers, *e.g.*, at Cawnpore Woollen Mills.

(5) Agriculturists who require power for irrigation.

The total **cost of development** includes the following particulars :—

(1) Foundation work, under water.

(2) Coffer-dams, and water control.

(3) Flood damages.

(4) Delays, due to floods and washed-out coffer-dams.

(5) General accident and damage account.

(6) Extras paid to contractor because of the first five items just given.

(7) Machinery erection costs.

(8) Details of construction and equipment—These mount up very fast and are frequently overlooked as unimportant.

(9) Engineering cost—7 to 7½ per cent. is a reasonable allowance.

(10) Damages to apparatus during erection and testing, and losses occasioned by them.

(11) Delays of all kinds—These mean increase in cost because of the fixed charges which continue until the plant is completed.

(12) Interest charge—This usually is figured on the basis of 5 per cent. per annum. As a matter of fact, the actual cash for a water-power development costs about 7 per cent.

(13) Contingencies—Even after a plant is once constructed, the contingencies are not removed.

Always be ready to pay, even unreasonable price, for guaranteed, quick completion. The extra charges for speedy work are, usually, small compared with loss of income, interest charges, possible flood losses and other costs that arise from slow process of construction. Successful operation requires quick construction and quick use.

A hydro-electric scheme which will pay only fair returns on a close estimate of cost, is seldom worthy of serious consideration as unforeseen expenses will often make it a losing investment.

Work out the technical features and see that :—

(1) The specified power is available.

(2) There is no unusual engineering difficulty.

(3) There is a fair chance of the market.

(4) Cost of all necessary land, expenses of buying out vested interest, cost of complete issue of concessions are reasonable.

Then make estimates of the following, remembering that plans are rarely made for large and important structures that do not require more or less modification during construction unless this fact is duly appreciated by designer and liberally allowed for in the estimates of costs—such estimates have always been found more or less inadequate to complete the structure. The hazards of construction increase with the difficulties of construction.

(*a*) Costs for concessions and land for the entire development, cost of buying out vested interests, etc.

(*b*) Cost of surveys, clearing of sites, loss of amenity, etc.

(*c*) Cost of providing all necessary construction plant, houses, and hutmen for construction staff and men, stores, offices, etc.

(*d*) Cost of making, relocating, and rebuilding roads and railroads to the hydraulic works for transport of materials, plant, etc.

(*e*) Cost of dam, bridge over spill-way overflow, scour, channel to permit of water-level of reservoir being lowered, gate towers, gates or valves, and operating gear, pipes, screens, racks, etc.

(*f*) Cost of constructing conduit, including cost of constructing diversion dam and headworks, with spill-weir, gates, screens, etc.

(*g*) Cost of constructing settling basins, syphons, aqueducts (if any).

(*h*) Cost of constructing forebay, including cost of allocated plant for concrete mixing, cost of gates, gate-house, racks, etc.; or, cost of constructing surge tank in lieu of, or in addition to, the forebay.

(*i*) Cost of pressure pipe line with anchor blocks and bolts, supports, bends, control gates or valves, expansion joints, air-inlet valves, venturi meters, etc.

(*j*) Cost of generating station building, machine shop, fitting and fixtures, etc.

(*k*) Cost of hydraulic and electrical plant, apparatus, equipment, auxiliaries, machine tools, piping, etc.

(*l*) Cost of constructing the tail-race, weir, etc.

(*m*) Cost of erecting dwelling-houses for the permanent staff and operators, stores, etc.

(*n*) Cost of legal and engineering charges, expenses of supervision, contingency allowance, and interest allowed during construction period, etc. Including cost of obtaining the authority of a Private Bill.

Make out the **estimated annual cost for** :—

(1) Redemption of capital.

(2) Renewal of plant and general working charges in connection with the hydro-electric plant, apparatus, equipment, etc.

(3) Allowance for depreciation and maintenance of works.

Remember this last item depends upon :—

(*a*) Quality of material used and the grade of engineering.

(*b*) Workmanship.

(*c*) Managment.

(*d*) Length, size, type, character and structure, slope of water-ways, velocity of flow, climatic conditions, voltage, factor of safety, etc.

(4) Insurance on buildings and plants and workman's compensation.

(5) Salary of staff and wages of operators, clerical and casual assistance expense of administration, general stores, oil, waste, and other incidental expenses.

Estimate of Cost Complete

A. Preliminary work :—

1. Surveys ... ... Rs.
2. Clearing sites ... ... Rs.
3. Roads, bridges, railroad, etc. ... Rs.

Rs.

B. Storage reservoirs (Brief statement giving name, location, area, available capacity for each of) :—

1. "a" reservoir ... ...
2. "b" reservoir ... ...

Rs.

C. Dam (Brief statement of length on the crest, height, type of foundation, etc.)

1. Estimated total cost ... ... Rs.
   (*a*) Cost per billion cubic feet ... Rs.
   (*b*) Cost per acre-feet ... ... Rs.

C-A. Weir ... ... Rs.

D. Conduit :—

1. Surveys and clearing route ... ... Rs.
2. Estimated cost for tunnels ... ... Rs.
   (*a*) Estimated cost for open-cut (lined) ... Rs.

E.	Forebay (Statement of approximate area; cubic contents of masonry or concrete required)	Rs.
E-A.	Surge tank	Rs.
F.	Pipe line (Statement of number, sizes, lengths, thicknesses, etc.)	Rs.
G.	Power-house (substructure) for ... units :—	
	1. Taking care of water, *i. e.*, coffer-dam, filling, face-planking, pumps and pumping, removal of ... cub. yd. of earth, etc.	Rs.
	2. Excavation of rock (cub. yd.) ...	Rs.
	3. Concrete ... cub. yd. at ... per cub. yd. ...	Rs.
	4. Steel ... lb. at ... per lb.	Rs.
		Rs.
H.	Power-house superstructure...cub. ft. at ...	Rs.
I.	Power-house plant :—	
	1. Hydraulic plant (complete)	Rs.
	2. Electric plant (complete)	Rs.
	3. Miscellaneous	Rs.
		Rs.
J.	Plant and tools	Rs.
K.	Tail race...cub. yd. at...per cub. yd. ...	Rs.
L.	Right-of-way, etc. :—	
	1. Reservoirs and flowage	Rs.
	2. Waterways	Rs.
	3. Concessions and land	Rs.
	4. Power transmission	Rs.
		Rs.
M.	Transmission lines : Telephone lines ...	Rs.
M-A.	Substations : Transformers, phase compensation, etc.	Rs.
N.	Permanent dwelling houses, etc.	Rs.
O.	General ; Legal : Legislative Bill ...	Rs.
P.	Engineering	Rs.
Q.	Contingencies	Rs.
R.	Working capital	Rs.
S.	Interest during construction	Rs.
	1. Operating expenses :—	
	Superintendent	Rs.
	Operators	Rs.
	Assistants	Rs.
	Labour	Rs.

2. Supplies ... ... Rs.
3. Maintenance and repairs, etc., of—
   (*a*) Dam, tunnels, open conduit, forebay, etc. Rs.
   (*b*) Storage reservoirs ... ... Rs.
   (*c*) Power-house and plant ... ... Rs.
   (*d*) Transmission and telephone systems ... Rs.
   (*e*) Substation and equipment, etc. ... Rs.
   Rs.
4. Depreciation :—
   The percentage of depreciation of the Chicago Sanitary District Hydro-Electric Plant has been shown under few heads from Electrical World, Feb. 28, 1906.
   (*a*) Dam at...per cent. on ... Rs.
   (*b*) Tunnels at...per cent. on ... Rs.
   (*c*) Conduit at...per cent. on ... Rs.
   (*d*) Forebay at...per cent. on ... Rs.
   (*e*) Pipe line at...per cent. on ... Rs.
   (*f*) Power-house substructure at...per cent. on. Rs.
   Building per cent. on ... ... Rs.
   (*g*) Power-house plant at...per cent. on ... Rs.
   (*h*) Transmission line (copper, insulators, towers, etc.) at...per cent. on ... Rs.
   (*i*) Transformers, etc., at...per cent. on ... Rs.
   (*j*) Telephone system at...per cent. on ... Rs.
   Rs.
   On other electrical appliances at 3 per cent.
   Pole line at 3 per cent.
   Water wheels at 2 per cent.
   Generators at 2 per cent.
5. Taxes, insurance, etc. :—
   (*a*) Government water-power tax ... Rs.
   (*b*) Land tax ... ... Rs.
   (*c*) Income tax ... ... Rs.
   (*d*) Insurance ... ... Rs.
   Rs.
6. Administration and general ... ... Rs.

Interest at...per cent. on total investment ... Rs.
Yearly cost of operation and fixed charges ... Rs.
Rs.

Unit cost of power :—

1. Cost per e. h. p. at the switchboard for total.........e.h. p. is Rs.——.

2. Cost per e. h. p. delivered at...for total............e. h. p. is Rs.— —
3. Cost per kW.-hour delivered at...for total......kW.-hour is.........(pie).
4. Cost per kW.-hour delivered, assuming a load factor of 25% ... (pie).

(a)	do.	do.	35%	...	,,
(b)	do.	do.	50%	...	,,
(c)	do.	do.	70%	...	,,

The cost of water-power depends on :—

(1) The cost of financing. This includes discount on stocks and bonds, interest during construction, cost of management and engineering and fixed and operating charges until the plant shall reach a paying basis.

(2) The investment in land, water rights, power plant equipment, transmission lines, substation, distribution system and other physical features and interest which must be paid thereon.

(3) On the loss on the depreciation of the various elements of the plant, the cost of maintenance and repairs, the cost of contingent damages from floods or other accidents.

(4) The operating expenses, including labour, oil waste, and other stations' supplies and expenses, including the patrolling and maintenance of transmission lines and distribution systems.

(5) Expenses for taxes, insurance, etc.

Approximate Proportional Investment Costs

Item of Cost.	*Percentage.*
Legal advice, preliminary work, and engineering reports ...	2·0
Lands, etc.	7·0
Discounts on bonds	9·0
Interest during construction	9·0
Legal and insurance	2·0
Construction of housing and camps ...	2·0
Construction plant	9·0
Coffer-dams and tracks	7·0
Dams	14·0
Power-houses	12·0
Machinery and apparatus	18·0
Head office expenses	2·0
Engineering and superintendence	5·0
Operators' dwellings, miscellaneous ...	2·0
	100

Note.—Many of these items of cost will vary with the type of development.

The annual **cost of developed power** includes—

(1) Administration and operating expense.

(2) Maintenance and repairs.

(3) Depreciation

(4) Interest, insurance and taxes.

The times vary with the duration and conditions under which the power plant is installed and operated.

Make a careful study of the available market and collect information on :—

(1) Class of power consumers—electrification of railways, tramways, industrial, lighting, irrigations or other types of load.

(2) Quantity of power required.

(3) Hours of operation—constant or intermittent—good or bad customers.

(4) Load factor and diversity factor.

(5) Proper and safe basis of respective power and energy sales.

(6) Mean unit selling price of energy at the consumer's terminals.

(7) Relative local generation production costs compared with the mean unit selling price of energy.

The **market price of water-powers** should be such that—

(1) The price at which the power company can afford to furnish power and insure a fair return for its investment.

(2) The price that the consumer can afford to pay and this depends upon what it will cost to produce the power by some other means.

The feasibility and practicability of a scheme are not entirely established if only the power and the market are available. We have yet to consider the question of Government Control and the State Laws regulating the fuel, land titles, etc., and in the case of hydro-electric plants the rights and the practicability of an economic development.

The investment balance, or the summation of the capital out-lay and the returns promised by the enterprise, the interest, sinking fund, cost of maintenance, operation, depreciation, taxes and insurances on the one hand and the receipts from sale of product on the other should be carefully considered. The balance assuming the total investment, interest payable during construction period, cost of lands, rights, franchises and legal and engineering services on the one hand and the debits for this balance such as fixed charges, interest, depreciation, administration,

taxes, insurance and operating costs should be carefully considered for determining the feasibility and practicability of a scheme. The larger capacity developments will show a greater surplus. If the investment balance shows a satisfactory profit, if you can afford to charge the lowest price per unit and supply energy free from all interruptions, your success is assured.

Presentation :—The engineer should sum up the findings from his investigation of the market, power capacity, feasibility, practicability and cost of the factory or installation and submit his report with such documentary proof and legal opinions as the occasion requires. The report should be exhaustive in every detail which is a link in the completed chain. It should contain all arguments, comparative costs and output data and the reasons why the recommended programme will guarantee the most resourceful and remunerative scheme. He should then give a description of the structural plan of the proposed works with brief stability proofs of the essential parts, and a concise specification of specially-adapted construction programmes and methods. He should present the detailed estimates supported by quotations of materials of recent date with delivery cost to site. The estimates should be made after the designs are completed and checked and should give details of every structure, their dimensions, weights, quantities, etc., and specifications of every class of structural material, proper qualification of labour and labour costs. It should conclude with a summing up of all the different materials required, grouped in accordance with proper classifications as to their character and cost. Add to the total cost the charge for superintendence, inspection, insurance and finally some arbitrary per cent. addition depending upon the thoroughness of the detailing of items for contingency allowance covering accidents, breaks, wash-outs, delays, etc. The quantities should always be quoted in positive figures, if possible.

It should close with the investment balance based upon the most remunerative scope which can now be definitely determined, and from this the cost of the product per Kilowatt-hour for any assumed load factor can be calculated.

It is not a wise or useful policy to suggest an alternative programme, because only the most feasible project should be advocated; nor should estimates be quoted in lump sums for power house equipments, dam, canal, etc. As the lump cost must be correctly known to the author of the report without his having developed it step by step through detail items, there is no reason why these should not be given in the report. Enter all calculations in a computation book devoted to that particular project. Every entry

should be clear and precise, each separate calculation should be given in a separate page and a comprehensive index for checking the items should be added. All sheets containing plans should be of uniform size, dimensions should be given in figures and identified by dimension lines ; the lettering should be clear ; location, plan, longitudinal and transverse section of all structural features with important details should be given.

Requirement of Investors :—The project must be carefully examined by experts of reputation—men who are of the highest ability and experience, who can and will vouch for the technical features of the construction, for the expenses involved, for the market available, for the legality of the enterprise and in fact for its probable complete commercial success. That is, the whole scheme should be carefully scrutinized by legal, financial and technical experts before undertaking the work.

The following books are recommended for further reading :—

Name of the Book.	Name of the Author.	Name of the Publisher
1. Hydro-Electric Development.	J. W Meares ...	Sir Issac Pitman and Sons, Ltd , London.
2. Hydro-Electric Survey of India.	Do. Do.	Superintendent, Government Printing.
3. Hydro-Electric Survey Triennial Report, Vol.III.	Do. Do.	Do. Do.
4. Hydro-Electric Report of the British Water Power Committee, July 1918.	... ...	... ...
5. Practical Water Power Engineering.	William T. Taylor	Crossley Lockwood and Sons, Ltd., London.
6. Hydro-Electric Power ...	Lamer Lyndon. ...	McGraw-Hill Book Company, New York
7. Economics of Engineering	L. D. Coueslant ... B. C. Chatterjee	Surya Narayan Chatterjee, Benares.
8. Water Power Engineering	Daniel W. Mead	McGraw-Hill Book Company New York.

CHAPTER III

HYDRAULIC LAY-OUT AND AUXILIARY STATIONS

Hydraulic Lay-out

The different types of falls can be utilized with slight re-arrangement of their course with the help of dams and reservoirs. Many falls are neglected which can be so developed into very useful water-power supply. The choice of type of development is frequently the greatest problem confronting the designer.

General :—The working head in any hydro-electric scheme may be developed in various ways, *viz.*—(*a*) The flow of water from a natural waterfall may be utilized by conveying the water through a pipe line from a point above the fall to a power-house installed on the bank of the stream below the fall. This may be either a high, a medium or a low head installation.

(*b*) A natural or artificial fall may be tapped and a stream or reservoir may be obtained from which the water may be conveyed in an artificial channel to a point from which a pipe line can be taken to a power-house at a lower level. This method is applicable to any head. The channel may consist of an excavated canal or a wooden, steel or concrete flume; allow the contour of the hillside at a slight gradient of from 1 in 500 and 1 in 1500.

The scheme to be adopted depends on the physical characteristics of the site, and should be such as to utilize the maximum possible portion of the head at the least cost, and at the same time to minimise as far as possible any interruptions due to floods and droughts.

In many cases of high head project it is often found that the quantity of water is not sufficient to meet the demand uniformly throughout the year. In these cases big storage tanks are required to maintain the continuity of service throughout the year. In smaller schemes in which the portion of the flow during period of light loads is impounded for the use in the period of heavy loads, construct smaller reservoirs for pondage.

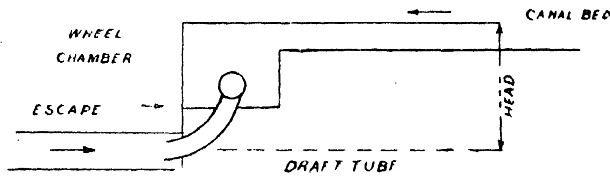
Types of Lay-out

The following types of hydraulic lay-outs are generally found according to the lie of the land :—

1. Lay-out of those depending on flow or storage.

(*a*) Natural waterfall lay-outs.

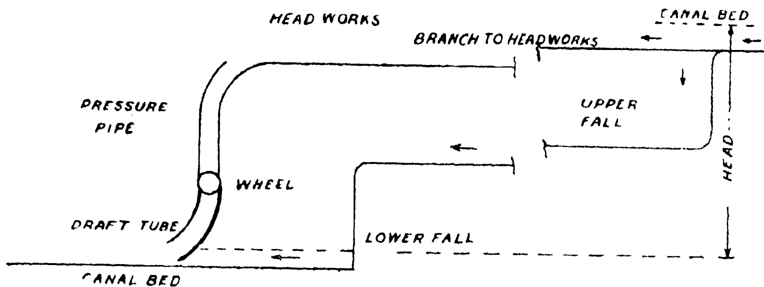
Fig. 1.



Low head lay-out canal fall with submerged wheel in chamber, may be on diversion of canal

(*b*) Lay-out with open channel.

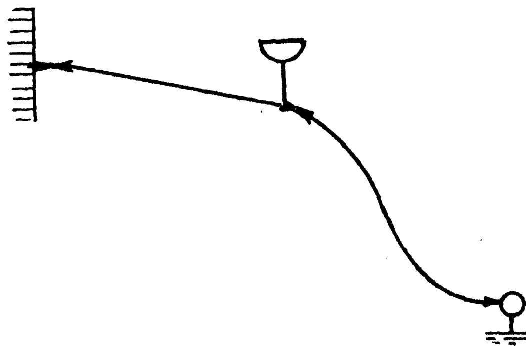
Fig. 2.



Low head lay-out for double canal fall with subsidiary head race channel

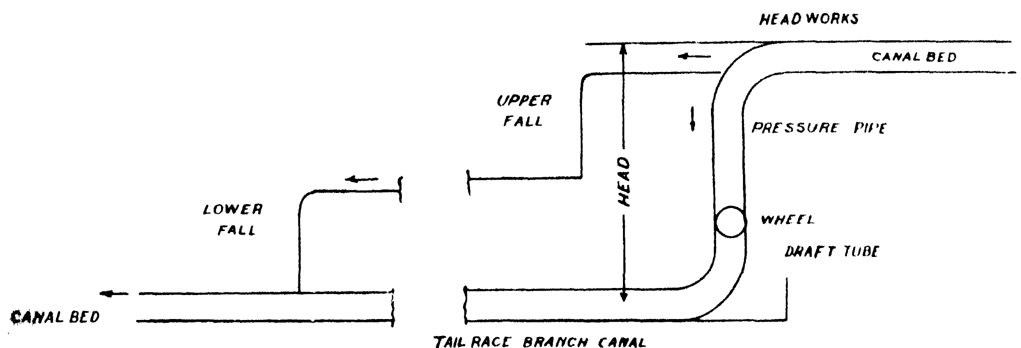
(*c*) Lay-out with surge tower.

Fig. 3.



Lay-out with surge tower.

Fig. 4.



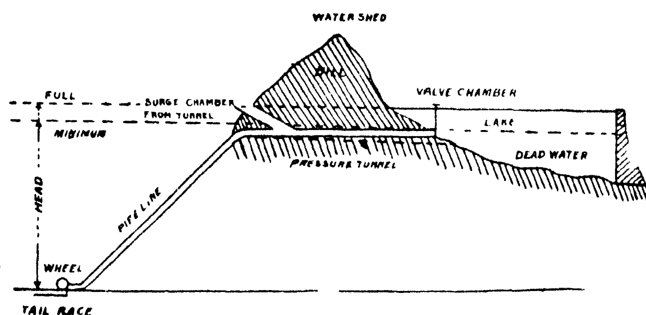
Low head lay-out for double canal fall with subsidiary channel, *e. g.*, Ganges Canal Installations.

(*d*) Lay-out with two rivers.

2. Those depending on storage alone.

(*a*) High head lay-out from storage crossing water-shed.

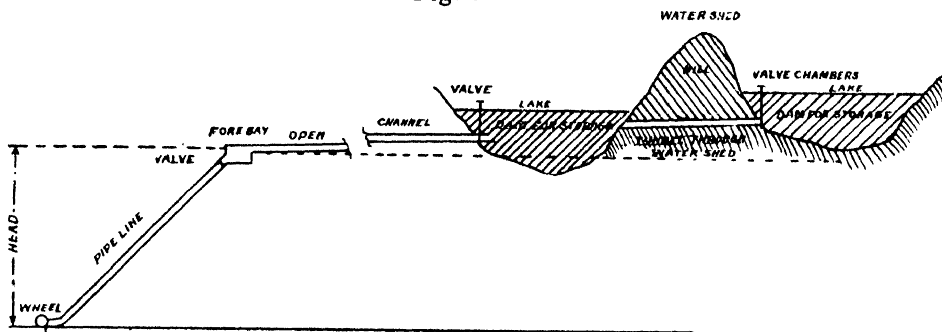
Fig 5.



High head lay-out working on storage with pressure tunnel connecting pipe line and lake—Example : Andhra Valley.

(*b*) High head lay-out with canal from storage on both sides of water-shed.

Fig. 6.



High head lay-out working on storage, with open channel from Lake to Forebay—Example: Tata H. E. P. S. Co.

3. Lay-outs due to bend in rivers.

Types of water-power development—Unshaped bend of Sutlej between Bhakra and Kirthpur.

Fig. 7.

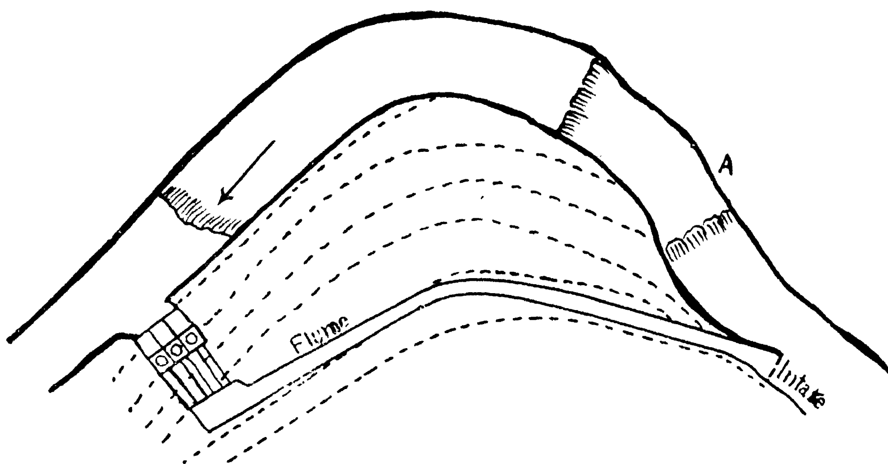


Fig. 8.

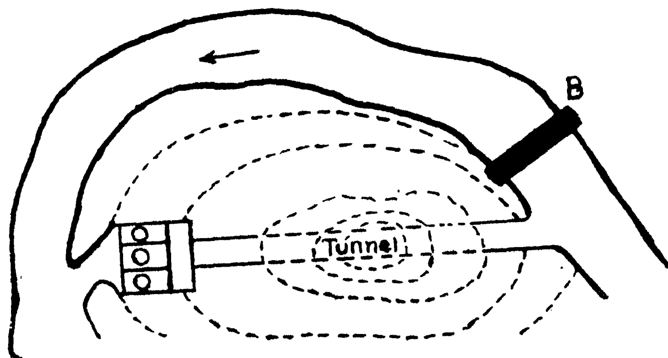


Fig. 9.

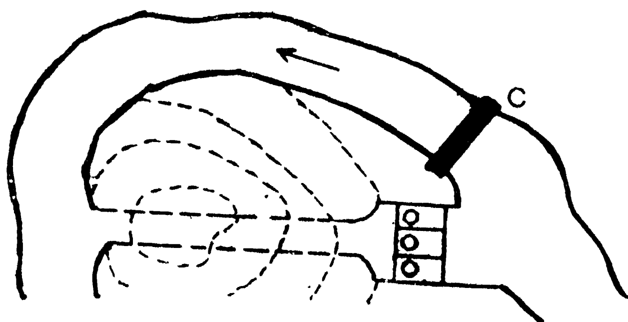
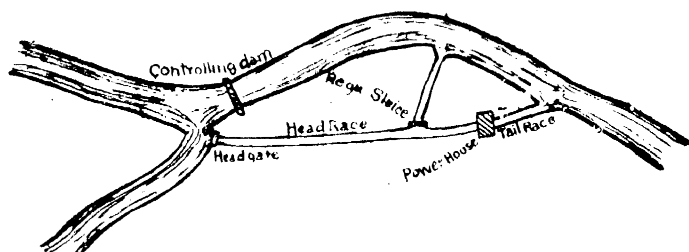


Fig. 10.



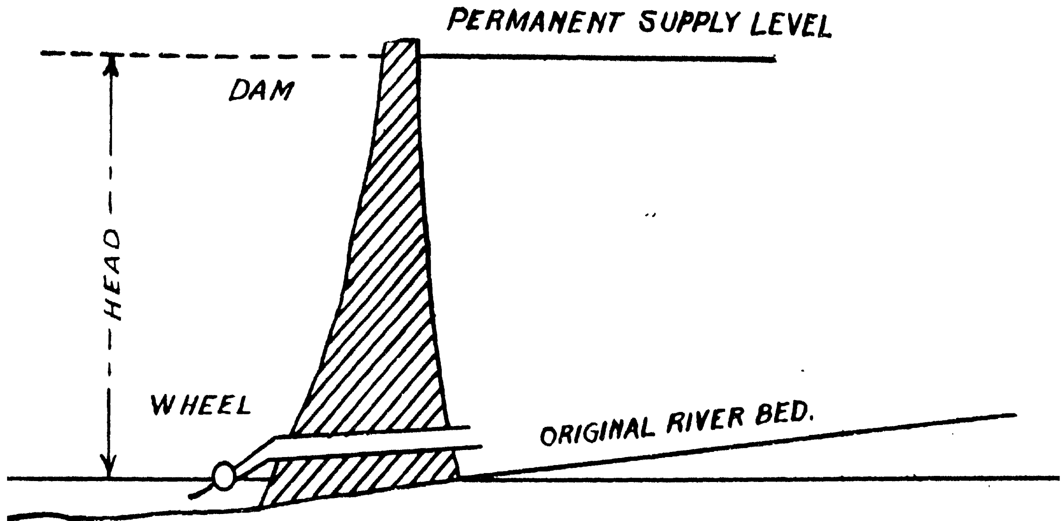
*The Government of India ordered a complete hydro-electric survey of the whole of India, and it is a pleasure to acknowledge that the work has been very ably carried out by Mr. J. W. Meares, C. I. E., Electric Adviser to the Government of India. The credit, however, of taking the first practical initiative in this direction belongs to His Highness the Maharaja Adhiraj Bahadur of Patiala, who had a skeleton project prepared by Sir John Benton, for harnessing the Sutlej river at a large horse-shoe bend some miles below Bilaspur."

4. Those depending on lifting dams for head.

(*a*) Lay-out with rising dam.

* Rai Bahadur Lala Rala Ram, C. I. E., I. S. O.

Fig. 11.



Medium or low head lay-out working on flow with dam to give only head or regulation. The storage not accruing drawn on. In some cases the same arrangement may offer sufficient storage to tide over short period of low discharge.

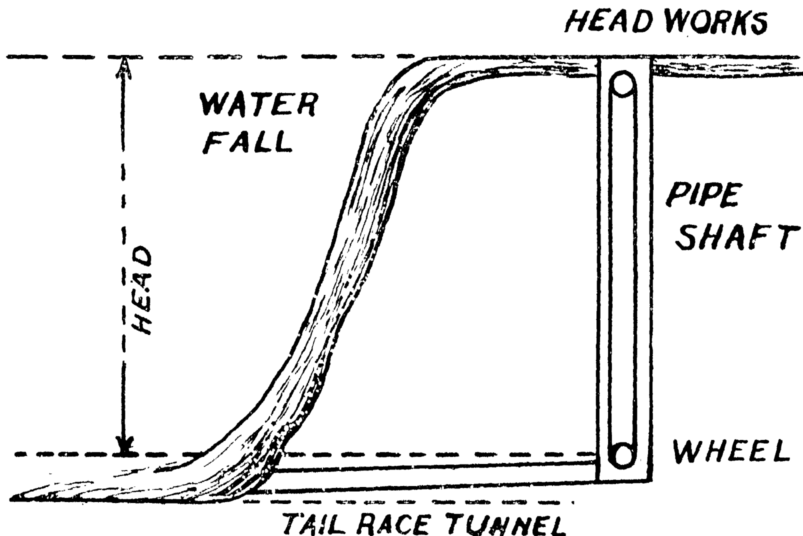
Classification of water-power :—

The basis of classification may be :—(1) according to nature of the water supply which may be perennial, or from reservoirs, or a combination of the two.

(2) According to the various methods by which the head or fall is obtained, *viz.* :—

(*a*) A natural waterfall with merely a pipe line conveying the waters from above the fall down to the turbines in the power-house below it. This may be a high, medium or low fall, fed by a perennial stream, or storage, or both.

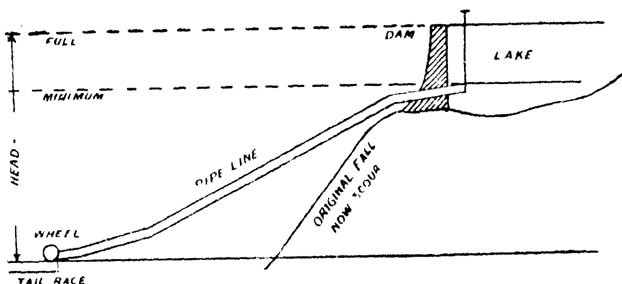
Fig. 12.
Natural waterfall lay-out with medium head.



Medium or low head lay-out of natural waterfall with vertical wheel shaft generator either in power-house under ground or connected by vertical shaft to surface—Example : Niagara Falls.

(*b*) An artificial fall created by a dam alone, the water being taken from the upper level to the lower level through the turbines.

Fig. 13.



High or medium head lay-out combined flow and storage—Example : Cordite Factory.

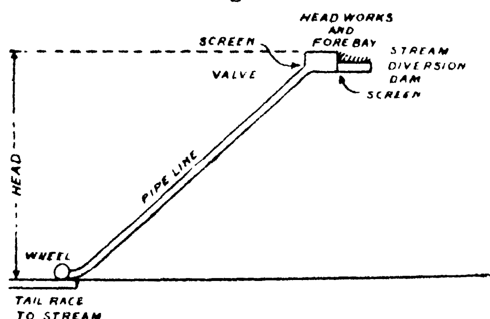
Sometimes the power station itself will be contained in the interior of a hollow dam.

Such an arrangement may be utilized for either a medium or a low fall. A succession of a rapid in a river is often dealt with in this manner, and a *canal fall is a special instance* of it also. The water stored hehind the dam will generally remain

at the highest level, constantly fed by the stream or canal, and if it is drawn upon, the fall available will soon diminish or even disappear. If, however, the water-spread of the reservoir is great, it will generally be permissible to use it for regulating purposes, as a few feet of depth may be sufficient when utilized to allow the whole incoming flow of the day to be used during the ordinary factory hours.

(c) An artificial fall, developed by tapping a perennial stream or a reservoir at a high level and conveying the water in an artificial channel to a point where pipes can be taken down to a power-house at a lower level. Very high heads are invariably so obtained, and the method is also applicable to medium heads and even to very low heads, such as combinations of two or more canal falls.

Fig. 14.



High or medium head lay-out with forebay on stream bank. No open channel, working on flow—Example : Mussoorie.

"There is no exact line of demarcation between these, except according to the type of turbine wheel generally adopted.

(a) A high head ; (b) a medium head ; and (c) a low head.

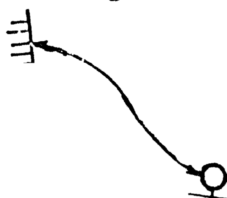
(a) *A high head* :—Thus high heads are those in which the Pelton type or jet impulse turbine is used, generally from 400 or 500 feet up to the limit of nearly 5,000 feet ; the flow of water required for a given power is here the minimum. High heads need the most careful treatment—Examples : The Tata, Andhra Valley, Pykara, Mussoorie, Nainital.

These may consist of the following types:—

(1) Headworks (or diverting dam) with pipe line direct to power station and no open channel.

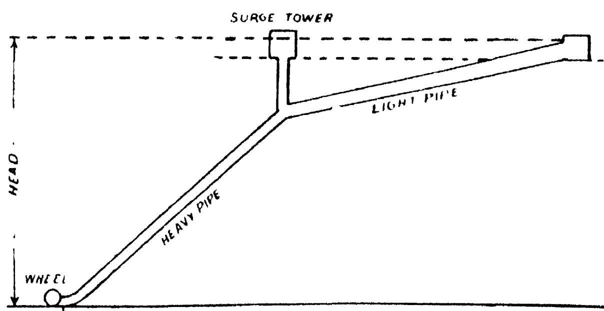
(a) with pipe line on a steep grade throughout, not involving a surge tower; or,

Fig. 15.



(*b*) with upper portion of pipe line on a gentle slope, probably involving a surge tower at junction with more vertical part of pipe line.

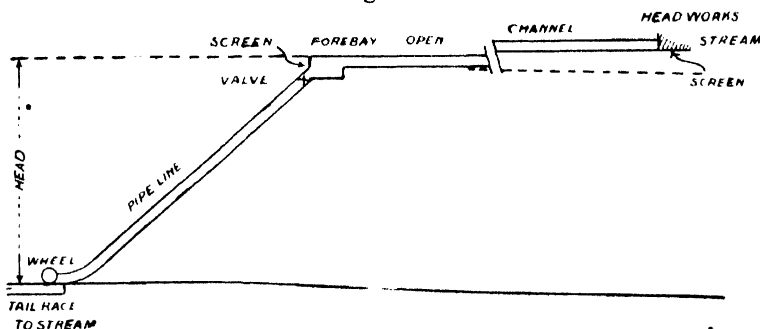
Fig. 16.



Pipe line with surge tower applicable to high heads with long length of moderate slope.

(2) " Headworks or diverting dam with open channel which ·ncludes tunnels to forebay from which pipe line will take off to ɔower station (*i*) with or without surge pipe, (*ii*) variation where :he channel will pass through a water-shed in a tunnel giving a ɔetter fall on the other side (*vide* Figs. 5 and 6).

Fig. 17.



High or medium head lay-out with one or more open channels from headworks on streams to forebay ; working on flow—
Example : Simla Municipality Installation.

(3) " High-head schemes are necessarily located in mountainous country and are generally fed from torrents of small volume, liable to heavy flood and consequently bringing down much mud as well as gravel and large stones. Even if the headworks have a dam, it is generally necessary to have settling tanks and a forebay, in order to prevent damage to the wheels from foreign matter travelling at high velocity. In order to get the most suitable fall with as short a length of pressure pipe as possible, it is almost always necessary to carry the water along the hillside for a considerable distance in an open channel, generally of artificial construction, though occasionally merely a cutting in earth. The channel offers a weak link in the chain, and duplication is impossible, though occasionally two or more separate sources of supply may be tapped by different routes. To allow for fluctuations of load (apart entirely from the question of settling tanks) a forebay at the head of the pressure pipes is essential, unless water is so plentiful that an excess is always running to the waste at that point. The forebay must be of such capacity that it will supply the excess water, over and above what the supply channel brings, at the time of peak load, or in the case of a sudden heavy loads, *e.g.* (short-circuits) thrown on the station. If the forebay is too small, and runs out even momentarily, air will get into the pipes and water hammer will probably follow when they refill." "Thus if the channel can deal with a steady load of 2,000 kW. and the peak load averages 2,500 kW. for 2 hours, there must be at least 1000 kW.- hours stored in the forebay. On 1,000 ft. heads this amounts to about 56,000 cubic feet. This amount, however, is no practical use as a reserve against a temporary break-bown in the channel or at the headworks, and wherever possible, a scheme of this nature should have enough storage at the pipe head to tide over at least a few hours' stoppage from such causes. For this much greater capacity is necessary, and it may not always be possible to provide it. If the ground will allow no more than a regulating forebay, a reservoir must be constructed elsewhere, and the nearer to the pipe head the better. If, however, large-scale storage by means of a dam is possible, it will almost always necessarily be either upstream of the headworks or at them, and the chances of a break in the flume must be risked. Large reservoirs, constructed by means of dams, can obviously only be located where the ground offers natural facilities, *i.e.*, at a comparatively narrow neck in a valley, which opens out above and does not rise steeply. Where comparatively small storage is required, more latitude is found. In some cases of steep ground it is possible to locate

the power-house so near to the headworks, even with high heads, that the water can be carried in pipes the whole way, without an open channel. Here the risk of break-down is a minimum. An intermediate case between these two extremes is where the distance from reservoir or headworks to power-house is considerable, but a channel cannot be constructed owing to the slope of the ground. Generally, the steep fall will be at the lower end of the pipe line, and the upper reaches will fall very little. In America the usual method of dealing with such problems is to use wood stave pipes for the upper part of the pipe line, with a surge tank at the junction of these with the steel pressure pipes, where the steep fall begins. The surge tank consists of a reservoir, elevated to the level of the water where it enters the stave pipes, connected by a large pipe (or separate pipes) to the junction. It not only serves as a regulating forebay, giving the water in the comparatively level upper section time to alter its velocity, but also it takes up and relieves all jars that might damage the weaker sections above it.

"For very high heads a comparatively small discharge is required, and this will sometimes be collected from more than one source and by more than one route. Earthwork channels can seldom be constructed in these cases, and the choice generally lies between galvanized iron (for very small discharges), masonry or concrete and timber. The first and last have the advantage that they can be carried on a structure without cutting away the ground, at least in the majority of cases. As landslips are always liable to follow the cutting of a roadway on a mountain side, this is a great consideration, and an inspection path can be carried alongside or over the channel. If masonry or concrete is used, water running down the hillside behind is stopped at the channel, and either flows into it with mud and stones or sinks in and causes slips. Catch-water drains of substantial size, leading to the nearest culvert, are in such cases essential. Smaller torrents may have to be crossed on bridges, and each of these should be used to afford a discharge outlet for the water in case of accidents and for scouring. Sometimes where water is not over plentiful, it will pay to divert these small torrents into the channel through a small settling tank; for, though their discharge may be negligible ordinarily, it may be sufficient in times of heavy rainfall to tide over a break-down in the channel higher up. Furthermore, when the main stream is in flood, the intake may have to be temporarily closed owing to floating debris, etc.; whereas the flood water of small supplementary streams will probably be clear and much collections of mud in the forebay may

thus be avoided. This is not always true, but is borne out generally in practice.

"Fairly low velocities, not exceeding $3\frac{1}{2}$ ft. per second in unpitched earth channels and 6 ft. per second in other kinds, are generally necessary for the preservation of the channel. The cross-section may be square, semicircular or trapezoidal, the latter being generally the best as giving the larger hydraulic mean depth. In order to lose as little head as possible (although a few feet on very high heads do not really count much), the slope should be kept small. The inter-relation between slope, velocity, dimensions, the construction and condition of the channel are stated in a number of rival formulæ, none of which are universally applicable. In practice the slope is generally decided on first—from 1 in 500 for small channels to 1 in 1500 or so for large ones—and the dimensions are calculated to suit. It is very necessary, however, to remember that any data found by a formula dealing with a straight length of channel will need alteration where there are bends or irregularities. An opening out of the width by an extra 5 or 10 per cent., carried back to the limit of the possible afflux, may be required to prevent overflow, or a corresponding raising of the height or increase in the slope. Uncontrolled overflow would be almost certain to produce landslides and break-downs. As, apart from faults in design, a small fall of earth may easily cause such an overflow, it is sound practice so to regulate the height of the side wall that overflow will take place most easily at places where it can do no harm, *i.e.*, over solid rock, or into streams passed by the way, or, in the absence of these, down artificial spill-ways.

"In order to prevent the channel being overfilled at the headworks a method often used is to place a diverting place in the fairway at a point near the headworks, such that any flow beyond the capacity of the channel is skimmed off and discharged back to the stream.

"Where masonry or concrete channels are used, most of the above considerations apply with equal force. Here it is necessary first to cut a roadway for the foundation, so that the liability to landslips and rock falls is greatly increased. Where the ground is at all doubtful, cut-and-cover is necessary, and it is better to spend money on revetments to begin with, than to take the chance of bad ground holding up. It is essential to make culverts under, or preferably over, the channel for the very smallest lines of natural drainage, both for safety and to prevent the access of stones to the channel. Often it will be advisable to lay small

drains down such places, with cross-collecting drains to prevent a wash-out.

"In extreme cases tunnelling may have to be resorted to, for the avoidance of bad ground. It may also pay at times to tunnel through a bluff rather than go round it. Tunnelling on a large scale is also sometimes required either to join up two sources of supply in different water-sheds or to lead water collected in one area to a different water-shed, where a more convenient or higher fall can be obtained. There are many examples of this class of work in America and elsewhere.

"With very high heads comparatively small pipes of great strength are required; the locality is likely to be far from rail-head, so that carriage to site is an expensive matter. It therefore pays as a rule to keep down the weight of individual loads, and to use a single pipe for each turbine. It is, however, often advisable to interconnect the various pipes at the forebay. Each pipe should be capable of being instantly closed at the forebay in the event of a bad blow-out, either by an automatic valve or by electrical control from the power-house. Each pipe must also have an isolating valve at the forebay or a gate in the forebay. Air pipes and filling valve may also be required.

"Where conditions are favourable, a single pipe is often used to serve two or more units by means of a receiver at the power-house, especially in the case of small plants, but this is not ordinarily recommended.

"In order to reduce the thickness of the lower sections of pipe it is customary to use two or more diameters, the smaller at the bottom and this also economizes in freight where they can be nested.

"Where individual pipes serve each wheel, it is advisable to interconnect them at the power-house, so that in the event of a joint blowing on one pipe while another generating unit is out of service, they can be cross-served. This involves extra expense in isolating valves, but is generally worth it. It is not an identical proposition with the receiver.

"Long lines of pipe, such as are required on high heads, require very substantial **anchoring** not only at the top and bottom but at several intermediate points. These also provide safe positions for expansion joints, which can be dispensed with only where the temperature variations are small or the line has many angles in it—and not always then.

"**Air Valves** (for Fig. see Ch. VII.) are not often required, but will be where there is an unavoidable rise in the pipe line. Occasionally air cushions are provided in very long lines, to take up

shocks ; at the Mill Creek plant in California the pipe is 8,700 feet long for a head of 1,950 feet, and both air valves and cushions are installed. The latter consist of duplicate cylinders for recharging at the necessary high pressure.

"The reduction of effective diameter due either to ordinary furring from lime, etc., or more particularly to fine mud deposited when the water is standing, may seriously increase the loss of head. Although cleaners of the revolving turbine type can remove a good deal of this deposit, the initial diameter should be always greater than is indicated by the velocity decided on."

Both high and medium heads development :—This consists of a diversion dam with an intake at the head waters. The flow is taken from this through tunnels, open canals or flumes to a forebay pond. The forebay is generally located on the hillside above the power-house and water is carried from the forebay through pipe lines to the turbines. The height being great, the quantity of water used is smaller than in low plants. Settling tanks are made near the forebay if the water requires them.

"Generally speaking, in the case of high and medium-head plants, a low diversion dam is all that is necessary, sufficient to ensure the possibility of getting the water into the channel or pipe under normal conditions, and also to protect the stream at this point from scouring out the channel and endangering the supply in times of flood. A few days' abnormal rainfall has lowered the level of a stream-bed 30 or 40 feet. For protecting the banks above and below the headworks boulder crates, made up of large boulders enclosed in a frame-work of heavy-gauge galvanized wire, have been found invaluable in every sort of river training works. Cases will sometimes be found where a permanent dam is not necessary, the natural lie of the ground alone ensuring the supply. Timber dams are sometimes used in America for levelling the bed.

"In the absence of a dam, whether the water is taken direct into pipes or into a channel—but more especially in the former case—arrangements must be made at the headworks for catching all debris. Generally, there will be a succession of screens or trash-racks through which the water passes to a small tank, in which any heavy matter coming through will deposit. This will need constant cleaning out, and should (in the absence of other reserves) be in duplicate. Nothing but fairly fine silt should pass beyond it. If there is no open channel, this tank should be large enough to remove even such silt as will deposit readily, or there should be an adjoining settling tank for this purpose. Where there is an open channel, there will always be the liability

of further debris being collected along it, and a forebay is necessary at the point where the pipes take off ; sometimes the lie of the ground makes it convenient to put intermediate silt tanks along the route. All these should be designed for rapid clearance at several exits, controlled by gates of substantial size. By sloping the floor from all points towards the exits the flow may, to a great extent, be utilized to carry off the deposited mud. The size of these settling tanks should be such as to reduce the normal velocity of flow to a few inches per second. Where there is a reservoir at the pipe head it is still necessary as far as possible to prevent the accumulation of mud in it, as a large volume will become so solid that its clearance is a matter of difficulty, and a silt trap immediately before it will assist in this direction. Where water is considerably in excess of actual requirements, it is possible to allow a proportion to run off continuously at the headworks chamber, through partly open sluices which will pass out all solid matter rolling along the bottom.

Medium or low heads :—Rapids, developed—

(*a*) by means of a headworks dam, to give extra head only (not for storage except regulation), with pipe line to power-house below rapids ;

(*b*) by a low diversion dam only, where the natural head cannot be increased conveniently, with a short pipe leading to the power-house below the rapids, but no open channel ;

(*c*) as in case (*a*) or (*b*) but with an open channel (which may include tunnels) leading to a forebay, from which a shorter pipe line will suffice. It may be many miles traversed by the stream in between the chosen headworks and the power-station sets. Here a canal or open channel is used owing to cheaper construction and greater care of working to carry the water on a very small slope.

This may consist of the following types :—

(1) Natural waterfall or series of cascades capable of development by direct pipe line from forebay at head of fall to the foot of the fall, with no open channel.

(2) Natural waterfall or series of cascades where bed slope of river enables extra head to be got by development.

(*a*) A surge pipe is seldom necessary on medium heads. Medium heads may or may not need complicated forebays.

A medium head :—Medium heads employ a variety of types of turbines, mostly impulse, but generally use the .pelton type or reaction turbine depending on the volume of water available and on local circumstances.

Example :—Gokak Plants, the Cauvery Fall.

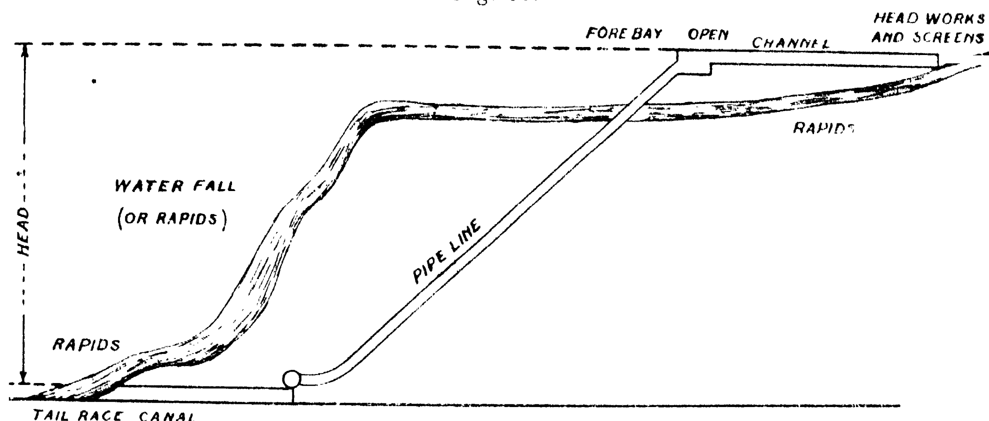
"Medium heads will generally be developed either from actual waterfalls or rapids in rivers, or from rivers, near the base of mountains where the volume is great and the slope comparatively small. Higher up-stream, it is generally possible to utilize a high head at less cost, even though a large volume may be available. Where many rapids occur in a short length of bed, it may be possible to pipe the water direct from the stream to the power-house with a forebay, interposed to prevent detritus getting into the pipes. In several American schemes either the whole power-house is located at the bottom of a shaft, cut down vertically in the rock behind the fall, or the turbines are placed in such a shaft and operate generators on the surface. Where there is an actual waterfall, as in many of the largest undertakings, it is seldom necessary to carry the water far to the forebay. Sometimes the pressure pipes can be led directly into the reservoir or into a forebay constructed immediately above the fall; a short canal at most is required, and as it must be of large carrying capacity there is a great saving in such a natural fall.

"Where the river has a considerable bed slope, or rapids, it may be necessary to carry the water for miles in a canal in order to accumulate a few hundred feet of fall; and if an earthwork canal is impracticable, an artificial flume is required, which adds both to the capital cost and to the liability to break-down.

"As the head decreases, and the volume of water increases, not only the flume but also the pipes increase in size, so that carriage to site often becomes a difficult and expensive problem where the installation is some distance from the railway. Occasionally a medium-head plant may be worked directly from a reservoir behind a high dam, but in such cases there will obviously be very large variations of head to contend with, sometimes from full head to none at all; sometimes—in fact, generally—such a reservoir will have been built primarily for irrigation purposes, so that beyond a certain minimum the flow is strictly regulated according to season. This detracts from the commercial value of any such scheme.

"While small works on medium heads may be served by the same methods as apply to high heads, canals in earth are generally necessary. The cost of an artificial channel would generally be prohibitive over a certain size. The largest timber flume was at Tacoma, U.S.A., and was 8 feet square and about 10 miles long,

Fig. 18.



Medium or low head lay-out of waterfall and rapids with open channel or tail canal Example: Cauvery Scheme.

with a fall of 7 feet per mile. At Cashmere the flume is 8′-4″ × 8′; 6½ miles long. No other material could then be used for a channel of this size, and timber has its draw-backs. Unless the water is running full all the time, warping is certain to occur, while boring insects and rot have to be faced. The large flume of the Jhelum Hydro-Electric Works has suffered severely in this way.

"There are no special points in which the pipe system on medium heads differs from that already dealt with. The pipes are of larger size, and their carriage becomes more serious unless they are built up at site, but on the other hand the locality is generally more accessible. Air valves and cushions are seldom required."

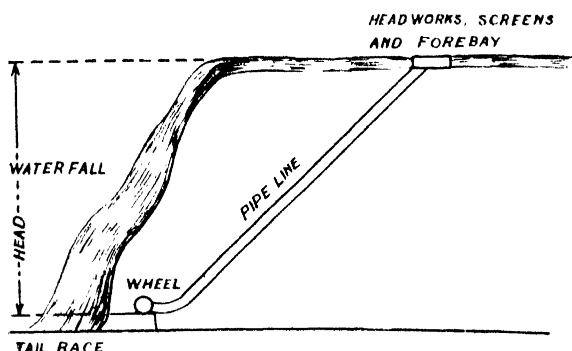
A low head :—Low heads (for examples the Jammu, Patiala, Bhatghar, Bahadrabad) are those in which pressure or reaction turbines are used, either completely submerged or with draft tubes, extending from the lowest practicable limit of about 1·5 feet up to about 80 feet and requiring a very large volume of water.

Special difficulties are met with owing to the great seasonal variation in the water-level; the monsoon rains, or even a local storm may cause a great rise in the level and it not infrequently happens that the rise is greater below the power-house than above it, so that the available fall is diminished. In narrow channels this is particularly likely to occur, so a gorge is generally a bad location.

Low head Developments :—This generally consists of a dam with its spill-way, the forebay, the intake and the tail race—The

dam which creates pondage at the point where water is to be *utilized*, so that water enters the turbine units by a short way while the quantity is large. With low heads a plain tank is generally sufficient. With low head plants the intake generally forms a part of the dam or power-house. Precautions are taken to guard against floating logs, debris, etc.

Fig. 19.



Medium or low head lay-out of waterfall.

This is available from:—

(1) A single canal.

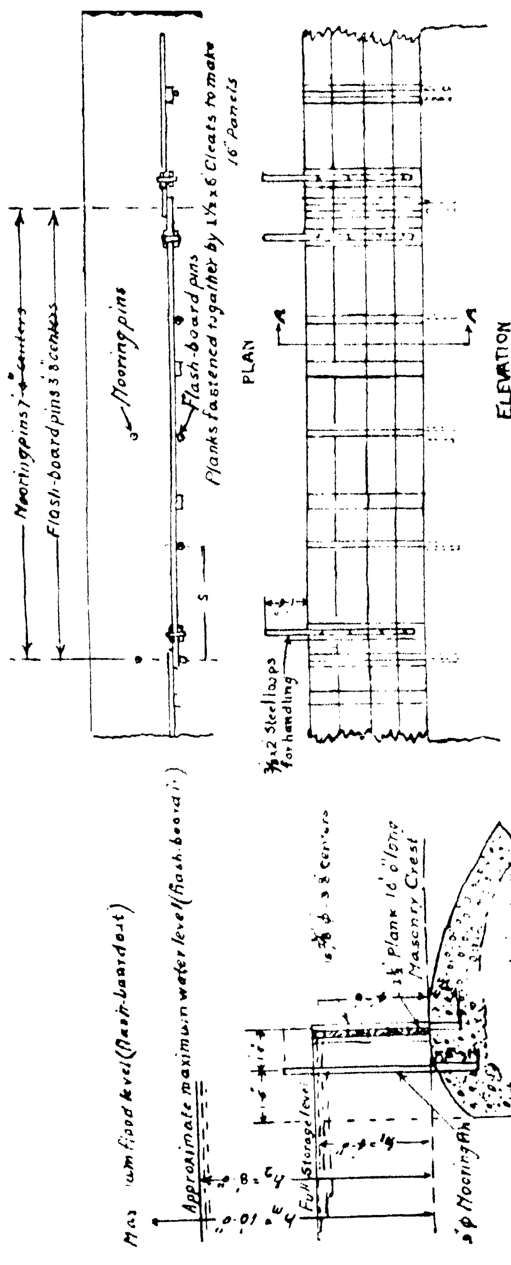
(2) Combination of two or more canal falls by means of subsidiary channel leading to a power-house near the lower fall or from the tail race of a power-house near the upper fall.

(3) A fall from a high level canal to a low level canal or from a river to a canal. Low head plants are liable to be shut down by flood water by reason of working and ceasing to exist necessitating stand-by fuel plant.

Floods and low water tend to decrease the power of low head developments, the former through lack of head and the latter through lack of water. The simplest way to maintain the normal head and thereby the full capacity of the water-power plant during flood periods (if not too great) is to apply some means for removing the high tail water, and it is possible to obtain a further increase in head by the use of flash-boards on the

dam (Fig. 20). In alternating current generation, the limit of satisfactory power, in times of flood, is reached, when the head of water is so far reduced that the generator speed drops below the permissive frequency.

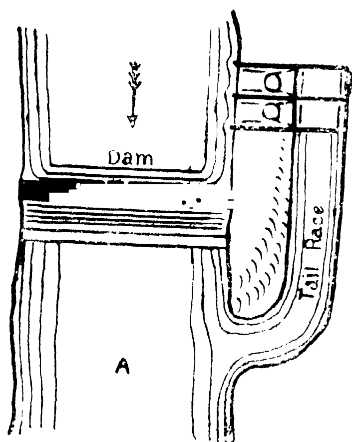
"Slow-running rivers and canals, with a small bed slope and no falls, cannot ordinarily be used for power purposes; as it is not possible to develop even a low fall without the risk of flooding the surrounding country above the dam; while the rise and fall of both head and tail waters, not necessarily to the same extent, further complicate the problem. Even where a small fall exists or can be made, these factors complicate the design.



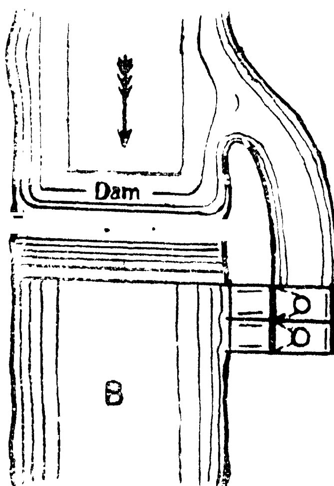
"In the case of irrigation canals, falls of from 3 to 10 feet are not uncommon, and can be developed by taking a feeder channel as a by-pass to the power-house, continuing as the tail race back to the canal below the fall (*vide* Fig. 21). Sometimes a subsidiary canal (generally in the form of a tail race) will enable two neighbouring falls to be combined so far as the turbines are concerned. Larger heads, but still of the low type, may occasionally be developed, where irrigation requirements permit, by discharging part of a high-level canal through turbines to a

low-level canal (*vide* Fig. 22). All irrigation canal projects suffer from the disadvantage that an occasional closure is necessary—whether for purposes of repair, or because the water is not required, or because it is more urgently required elsewhere."

(*a*) Lay-out with dam and tail race.
Fig. 21.

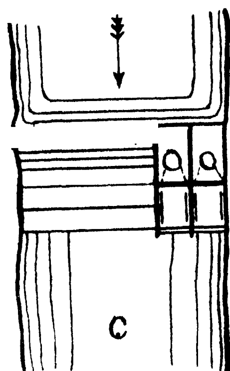


(*b*) Lay-out with dam and head race.
Fig 22.

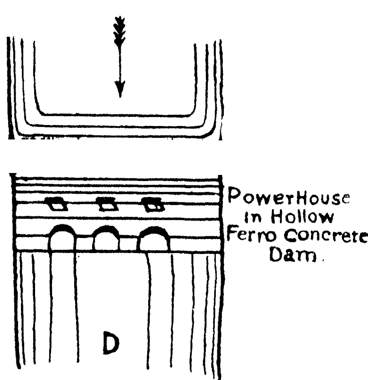


Types of low head developments. *e. g*, Ranbir Canal at Cashmere.

(*c*) Lay-out with power-house in hollow concrete dam.
Fig 23



(*d*) Lay-out with power-house in the dam itself.
Fig. 24



Dam and power-house as at Bahadarabad Plant.

We find "A further possible element of risk (in power supply) is an interruption in the canal supply. Our Nerve Centre of power is at Bahadarabad, and all our works up to Dhanauri are now so strong and safe that danger is hardly probable so far as human ingenuity can foresee. At Dhanauri we can escape all the water passed through Bahadarabad falls."*

* P. VII, The Ganges Canal Hydro-Electric Scheme.

"In the case of rivers, low falls may be developed on the lines already described for medium falls—always bearing in mind the ever-increasing size of channels and pipes as the fall decreases. In addition there are various schemes in which a dam is placed across a river not so much with a view to storing water as for the purpose of obtaining head. Here the water is fed directly from the dam to the turbines, which, in some cases, are placed inside a hollow dam.

"Great variations in the level of the head and tail waters are often a feature of low head river developments, and floods are a serious menace. In some cases, *e.g.*, at the station of the Portland Railway Light and Power Company, Oregon, and at Bhatgar Dam Plant in Bombay Presidency, double turbines are installed, one for the maximum head and the second for the reduced fall. Flood gates, as used in irrigation canal headworks, are required to deal with abnormal rises.

"In the case of low heads the water is generally clear, as low velocity postulates a small bed slope and deposition will occur in the river itself. In any case the quantity to be dealt with is so great that artificial settling cannot be arranged for. Strainers (known in America as trash-racks) are necessary to prevent floating matter entering the pipes, and a forebay may be required to catch anything that may pass through, as well as for regulation. A long canal is seldom necessary, and the forebay will either be at the side of the fall, parallel with the riser, or across it on or in a low dam. It depends on the ground whether a diverting dam or anicut is required or whether a forebay can be opened directly into the bed of the river. In the majority of cases of very low heads the power station is built across the river, and the water flows directly through the strainers into the turbines.

"With very low heads and submerged turbines the draft tube is the only pipe required, while with somewhat higher heads there will be both pressure pipe and draft tube. With large units of plant the size of the pipes becomes very great and the internal hydraulic pressure becomes a secondary consideration. Cases are on record where an automatic gate, serving a very large pipe, was closed at the forebay and air pressure caused a complete collapse. No interconnection is usually practicable or necessary with low heads."*

* Extract from a paper on "The General Principles of the Development and Storage of Water for Electrical Purposes"—from the Journal of the Institution of Electrical Engineers, Vol. 57, No. 283, June, 1919.

Auxiliary Stations

Object :—The object of hydro-electric power development is to produce power which will give the greatest return on the investment necessary for such development.

"The greatest" has reference to (1) the net profit from the sale of power, (2) increased return from the developments that may be benefited by its installation. For example, reservoirs are constructed for such development and the tail water is utilized for irrigation purpose in an area where there was no irrigation before, and thus increase in the produce is due to the development.

The factors which influence the cost of development are :—

(1) Capital invested for such development and the interest thereon and depreciation of the plant.

Power plant, of every kind and description, is regularly written down on the basis of its probable life (say 15-30 years). Hydro-electric lay-out remain indefinitely serviceable with proper maintenance. The over-all rate chargeable on the whole project will, therefore, be a much smaller percentage than it is in fuel-driven installations. But there is often a very large initial expenditure such as is necessary to construct headworks and channels which are unproductive until the plant has been extended to its full capacity. For instance, in the case of dams by far the greater part of the cost is in the foundations and base and there would be little saving by not building in the first instance to the height ultimately required when the full capacity of the plant is utilized.

(2) Operation, maintenance and repair cost.

(3) Taxes, insurance, management and other overhead charges.

(4) The amount and duration of the various classes of power produced and utilized.

(5) Reliability and class of service as affecting the sale value of the power.

(6) The length of the non-producing period of construction.

(7) Auxiliary stations.

All these factors are interrelated. It will be seen that any change in design must affect some factors favourably and some adversely. Hence the designer must see that the design must be such as to secure the greatest efficiency, economy, safety, reliability, facility for management and output.

In the economic design of hydro-electric stations a number of conditions, types of apparatus and structures are assumed.

Such assumptions are based upon a careful consideration of local conditions, the cost of power and development, the nature of the market and factors affecting economy, adaptability, successful operation, etc. Sometimes auxiliary plants much increase the capacity of the plant—*vide* pages 22 and 23.

Reserve :—In the case of hydro-electric installation supplying light and power for general distribution in a city a reserve steam or gas plant is often installed and is kept ready for immediate use at any time in case of an accident to the main plant owing to a serious reduction in its output due to drought, flood or ice troubles on transmission line. Often the two make a combination superior to other by itself, such as at Saharanpur, etc.

Where storage is expensive, the combination of steam and water-power plant often forms the most economical method of using water-power. Full economic advantage of the stream even with storage regulation, cannot be secured without a source of auxiliary power.

Auxiliary stations may be divided into four classes :—

(1) **Stand-by Stations** :—To take care of the load in case of break-down to the hydro-electric machinery or the transmission lines. The size is determined by the load which must be maintained under all conditions. The capacity of the auxiliary plant should be such as to guarantee the continuous power output which may be realized from the maximum hydro-electric equipment by supplementing deficiencies caused by flow and fall reductions.

Location :—Close to the important distributing centres. For large and extended systems it is better to provide two or more distributed stations.

Starting :—A quick start is an essential requirement of an emergency stand-by station. It is, however, not customary to have all the boilers under fire to take over the load immediately in case of an interruption. Some of the boilers are, as a rule, kept under banked fires part of the time to secure the most important load, and the generations are operated as synchronous condensers to improve the power-factor of the entire transmission system, which may carry a large inductive load.

Under these circumstances it is particularly easy to respond to sudden load demands, because the unit is already up to speed, and in synchronism, the turbine is kept warmed up, and only a change in the field excitation is necessary to place the unit on the line, which takes only a few minutes at the most. When storms are approaching, the entire reserve equipment should be made ready to respond immediately to any emergency that may arise.

(2) **Low-water Station** :—The function of the auxiliary plant, when used as supplemental capacity during low-water periods, is similar to that of the storage reservoir. It is also of value in making up storage of water-power from loss of head during high back-water caused by floods.

The problem, therefore, really resolves itself into two questions. First, in the case of a plant already in operation, to what extent shall an auxiliary supply be provided to convert the variable power supply into a continuous supply? Second, in case of a new development, for what capacity shall it be built?

"In order to get the best economy out of the steam station it must operate at practically a constant load corresponding to full load on one or more units. In order to get the best economy out of the water-power station with the water available during low-water periods, the highest water level attainable—in other words the maximum head—must be maintained at all times.

"It is impossible to conform to both of these requirements, especially where the minimum stream-flow capacity and the steam-station capacity combined are not sufficient to carry the peak load. In this case the steam plant can be operated at practically a constant load, using the water-power during the peaks and storing water during balance of the time. With high-head plants the head gained by storage is not of importance; so that the steam plant can be operated most economically on constant load, allowing the water-power to take the peaks. With low-head plants having considerable storage capacity, both plants can be operated advantageously during the low-water period. Here again the water-power should carry the peaks, and the steam-plant should be operated at constant load over a sufficient part of the day so that the water-level will not be materially affected. This method of operation will prove much more economical, both as regards fuel used and labour required, than the method of carrying heavy loads on the steam plant during the peaks, thereby requiring more boilers and machines in service and, consequently, more fuel and operators.

"The term 'peaks' is intended to cover heavy load period of the daily load curve, and not the momentary load fluctuation. Assuming equal governor or speed regulation and equal flywheel effects, these momentary load fluctuations are divided between the stations in proportion to the total capacities of the generators operating in each station. The flywheel effect of the steam turbine is usually the larger and steam turbine governor is the more sensitive. The steam turbine station will, therefore, ordinarily take more of the momentary fluctuation than its proportionate capacity in operation.

"Some fuel can be saved in developments of this kind by carefully observing the rainfall within the drainage area of the stream-developed. In case of rainfall within this area, the stream plant can be shut down immediately and all the load taken over by the hydraulic plant at the expense of reducing the level of the reservoir. The increased stream-flow will again fill the reservoir. Rainfall at the head waters of a large stream would not materially increase the stream-flow at the development for some time; and, consequently, a considerable saving in fuel would thus be effected. During the dry season, water flowing over the dam means fuel wasted; and, therefore, if enough reliance could be placed in weather forecasts to anticipate rainfalls, the steam-plant could be shut down in time, so that the reservoir level would be reduced sufficiently to take care of the increased flow without wasting any more over the dam than necessary."

The auxiliary plant will be rarely called for until the hydro-electric plant has been in operation for some years, but in estimating upon the whole project and deducing the net earning capacity of the proposed enterprise this factor must be included as an item of investment of maintenance, depreciation charge, and of operating cost.

No station should be without some electric storage capacity for station operating purposes such as are required in the regulation of the voltage on exciter outlets, operating on high tension switches, and lighting the station.

The *location* of a steam auxiliary should be decided largely by the conditions as to economical fuel supply. The auxiliary plant units must always harmonize with those of the hydro-electric installations.

Care of Auxiliary Plant:—Whether its services are required or not, it must be run periodically and the charges to be debited to it add very appreciably to the cost per unit of power sold.

(3) **Peak-Load Stations**:—The function of this is very similar to that of pondage above the water-power plant in carrying the daily peaks of load on the system. It increases the operating load factor and thus the output from water. The total output of the system to be carried by the auxiliary plant determine the first cost and relative economy of generation.

(4) **Base-load Stations**:—These plants operate continuously the water-power being supplemental to the steam-power. Low operating costs are essential for this type of plant. The auxiliary plant units should always harmonize with those of the hydro-electric stations. **The capacity** of the plant is decided by the

magnitude of the load which must be maintained under all conditions by supplementing deficiencies caused by flow and fall reduction and should be prima facie at least of one generator but in this respect may determine the unit question. The predominative factor in deciding on the **type of plant** should be low capital cost rather than low operating costs. The design of the plant should be such that it can rapidly take up the load on emergency.

Examples of Stand-by Stations

* "The Moradabad City and civil lines supply, on the other hand, is A. C., and in view of the importance of this town and its distance from the nearest power station (111 miles) with the treacherous Ganges Valley tract intervening, A. C. **stand-by plant** was provided in addition, *viz.*, two Sulzer Metrovick 250 kW. generating sets. Two B. T. H. 250 kW. rotary convertors are provided for converting the railway load which may reach a maximum of 250 kW.

A 3,300/11,000, 500 kW. transformer has been provided for the branch line to Chandausi and Sambhal, two important towns in the Moradabad District.

Two 3,300-volt underground cables connect the two indoor sub-stations for the Moradabad City and civil lines supply respectively.

At Saharanpur, bulk supply is given on a separate agreement to the local distributing company whose existing oil engine D. C. plant 250 kW. capacity was purchased at a cost of Rs. 1,10,000/- by the Irrigation Department as a stand-by. An outdoor sub-station steps down the 37,000-volt hydro-supply to 11,000 volts through a bank of 3 single-phase 312 K. V. A. transformers feeding the 11,000-volt branch line to the Saharanpur District and two departmental 250 kW. rotaries (by the English Electric Co.) for the supply company's 400-volt D. C. supply including the North-Western Railway load.

Similarly, at Aligarh, a commercial agreement has recently been signed by which the department takes over the local supply company's present 3,300-volt generating plant (450 kW capacity) as a *stand-by* and supplies hydro-power in bulk from the Sumera line which passes through Aligarh by means of a 750 K. V. A. 37,500/3,300-volt transformer, which will also supply current to the Muslim University (peak load 100 kW.) and to the East Indian Railway, (50 kW.)."

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*Page 19, The Ganges Canal Hydro-Electric Scheme, by W. L. Stampe.

Interconnection of Systems

These enable the individual systems to supply power to each other on an "interchange" contract basis. The peak loads of the different systems may not coincide, the minimum stream-flow may occur at different times on the different water-sheds, common steam reserve stations may be used, and, in general, the operation may be so improved that a most efficient and reliable service can be rendered to the customers of all the systems so tied together.

Groups of established systems, although located in vastly different localities, may be brought together under one holding company; and the creation of such companies may, in many instances, be attributed to the high-class service and financial success of our small and medium-sized light and power systems. The economies due to a central management, the benefits of the best technical and expert advice, applied even to the smallest central station, the cumulative effect of active, up-to-date new-business campaigns at every point, all have contributed to an improved and cheaper service to the consumer; and without the facilities of such a control they could exist only in the larger communities. Another very important advantage is the great problem of financing all these undertakings and providing funds for extensions to meet the ever-growing demand of the public for electric service. It is possibly in providing ready financial facilities for these purposes that the holding company performs its most important function.

Complete metering stations at the points of interconnection provide records upon which rates and charges for exchanged energy are made.

Interconnection may be justified for financial advantage alone, but the basic reason for interconnection should be for the ability to render intersystem assistance during local troubles and in that way to prevent interruption of service to the customer even though no definite financial expression may be given to it.

When power plants are operated in connection with an electrical transmission system which permits of the following desirable advantages—(1) transfer of load from one river to another, (2) transfer of load from one plant to another, (3) transfer of load from one point or locality to another—then and then only can each stream be operated at its maximum efficiency.

CHAPTER IV

HYDROLOGY

Sources of Water Supply :—The precipitation in the form of rain or snow which reaches the earth is the ultimate source of our water supply.

Rainfall :—Rainfall of the world is about 36 inches per annum, and it varies with the situation, physical configuration and direction of the prevailing winds. Thus it is zero in the arid regions, while at Cherapunji it is about 500 inches.

Generally, there is the monsoon period and the dry period, but in the South there are two distinct monsoons, the South-West and North-East.

The rainfall in India is seasonal and during the greater part of the year occasional. There are 8 dry months and 4 wet months. As a result many of the rivers and streams in India almost dry up before the end of the dry season. In such cases development by the monsoon storage is the only method of getting continuous power, and this is not always possible, although discontinuous water-power may be of value in particular instances for utilization in industries that are only manufacturing during the rainy season or for any purpose in combination with reserve plant driven by steam or other fuel.

It may, however, be accepted that a study of the daily rainfall observations over a period of 30 years will give a fairly accurate idea of the conditions which are likely to occur in future, and if different percentages of run-off are assumed for light, medium and heavy falls, depending on the nature of the catchment, a fairly accurate forecast can be made. At any given place the total rainfall during any period of fifty years will be within about 2 per cent. of the total rainfall at the same place during any other period of fifty years, while the rainfall of any period of twenty-five years has been found to fall within about 3·25 per cent. of the mean of 50 years.

In India the rainfall varies in different localities from 450 to a few inches, mostly precipitated, in about 4 months, while catchment areas vary in nature from waste sand to steep rocky country.

It is now recognised that there is a nearly constant ratio on any given area exceeding about 1000 acres, between the true mean annual rainfall, the rainfall of the driest year, the two driest consecutive years, and any other group of driest consecutive years.

Investigations apparently have proved that with the introduction of forest growth, rainfall that have ceased may be made to start again, and by hastening most kinds of tree growth on desirable catchment areas, it is very likely that rainfall will be increased and be made more regular. The literature on the influence of forests in relation to rainfall, evaporation and stream discharge is voluminous, and it is now definitely established, beyond possibility of dispute, that the removal of forests in conjunction with the resulting soil erosion tends to cause permanent streams to become seasonal. Between the years 1843 to 1883 the Ekaterinosalv Government, Russia, cultivated a forest of 5000 acres, and established two meteorological stations in this section. Reports show, that since the introduction of the forests, showers are much more frequent, and the previously-feared dry seasons are things of the past. It is a well-known fact that the soil of forests retains the water of precipitation more uniformly, and releases it gradually, so that during dry season a supply of water is more assured.

For normal rainfall *vide* "Memoirs of the Indian Meteorological Department, monthly and annual rainfall normals."

Variation in Rainfall :—The rainfall varies greatly in different countries depending upon the geographic or topographic conditions of the country. A knowledge of the maximum rainfall is essential for determining the discharge, but for practical purposes the average monthly or the monthly of the driest year both affect the supply..

	Monthly Rainfall in Inches.												Total annual.
	J.	F.	M.	A	M	J.	J	A.	S.	O.	N.	D.	
Madras (mean of 33 years).	91	·33	·18	61	1·08	1·90	4 03	4·94	5 14	11·27	12·78	6·24	49·5
Bombay (mean of 39 years).	·07	·03	·01	·03	·51	20·88	28·00	15·95	12·50	2·00	·34	·06	80·4
Cherrapunji (mean of 40 years).	·69	2·02	12·12	33·07	43·84	95·72	99·54	76 09	47·41	13 83	1·45	·20	426·0

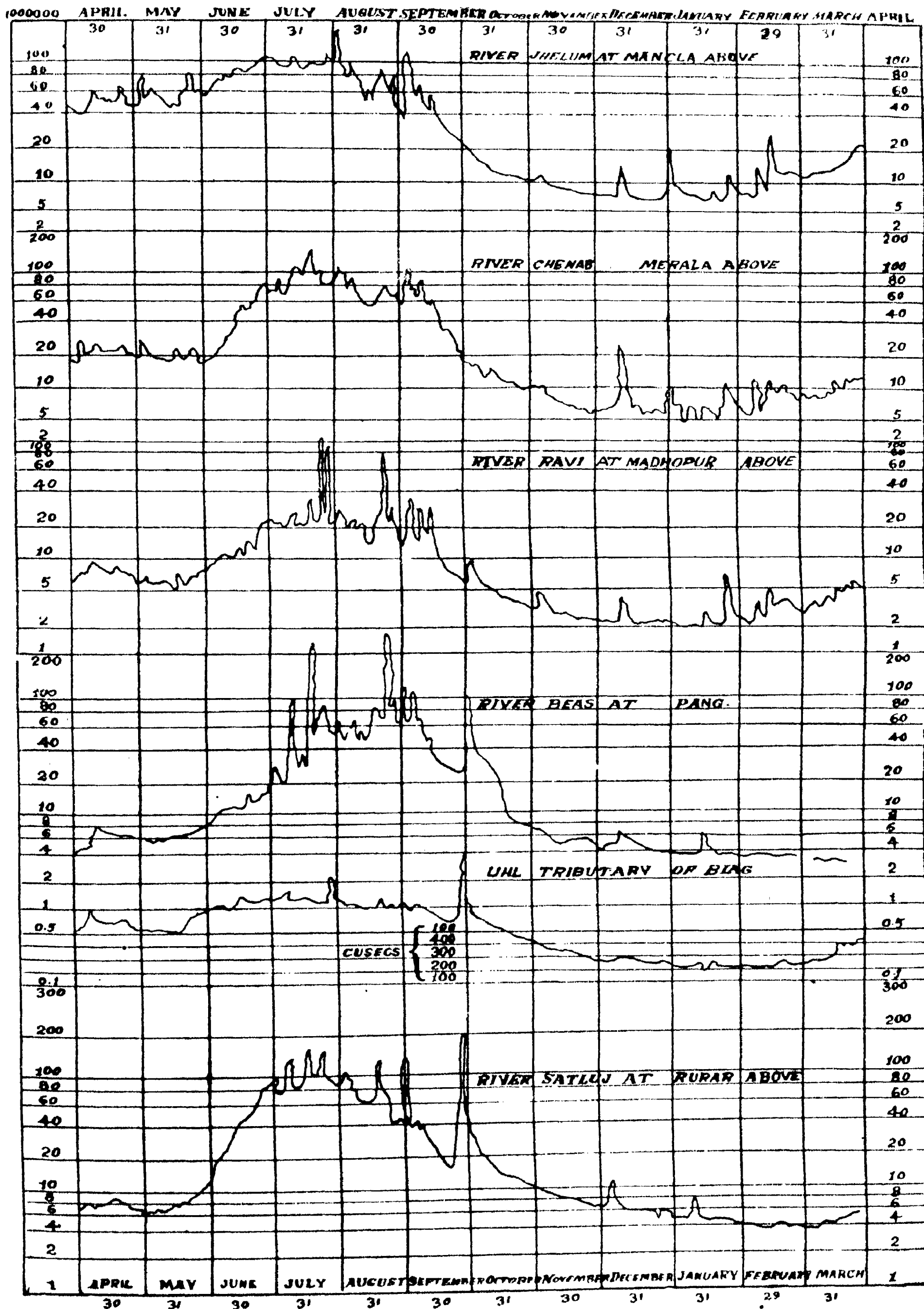
For calculation of available power the basis should be stream gauging as stream gauging is much more important and more reliable than rain gauging or rainfall data.

The Study of Stream Flow from Their Hydrographs

The extent of stream flow from a catchment area depends primarily upon the total area of the basin drained, the depth of rainfall or total precipitation, as the elements of rain, snow, etc., the geological formation, the topographic features, and also the general characteristics of the area.

The variation of stream flow is best studied from their hydrographs. These diagrams indicate the rate of flow prevailing during every day of the water year, and are prepared from their gauge readings. The ordinates represent the daily flow in cu. secs. at the point of observation and the abscissas are the elements of time. Hydrographs are extremely interesting and indispensable to determine correctly the water storage for the supply of water during the dry season, and are useful also for comparison with observation made at other points in the same river. Note that if the ordinates are changed after taking into account the fall of the river, the diagram may become a power curve. To be of any efficient value hydrographs must be plotted for a long period of time.

SUPPLY DIAGRAM SHOWING DAILY DISCHARGES OF PUNJAB RIVERS.
FOR THE YEAR 1924-25.
(In thousands of Cusecs.)

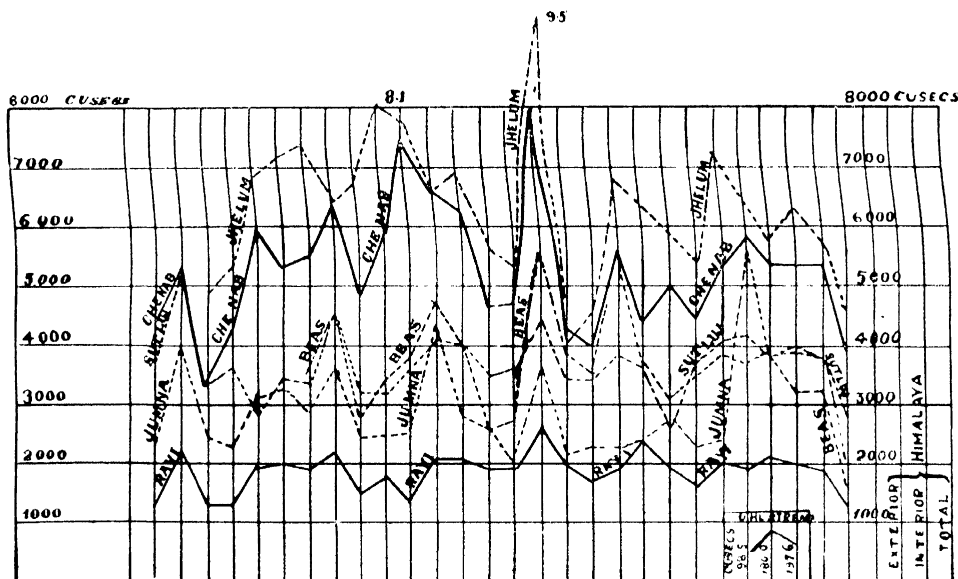


STATEMENT AND DIAGRAMS SHOWING

MINIMUM DISCHARGES OF PUNJAB RIVERS FOR 20 YEARS.

(FROM 1899-1900 TO 1924-25)

AND OF UHL STREAM FOR 3 YEARS (1922-23 TO 1924-25).



YEARS	1899-1900	1900-01	1901-02	1902-03	1903-04	1904-05	1905-06	1906-07	1907-08	1908-09	1909-10	1910-11	1911-12	1912-13	1913-14	1914-15	1915-16	1916-17	1917-18	1918-19	1919-20	1920-21	1921-22	1922-23	1923-24	1924-25	(a) 1922-25	(b) ON RECORD	PERCENTAGE (b) TO (a)	CATCHMENT AREA IN THOUSANDS OF SQR. MILES: EXTERIOR	INTERIOR	TOTAL
JHELUM			4·8	5·3	6·8	7·2	7·4	6·5	6·7	8·1	7·8	6·6	6·9	5·7	5·3	9·5	4·0	4·5	6·3	6·4	6·0	5·4	7·3	6·5	5·8	6·3	5·8	4·5	73 o/o	7·0	8·0	16·0
CHENAB	4·3	5·4	3·3	4·2	6·0	5·3	5·5	6·4	4·8	5·9	7·5	6·6	6·3	4·6	4·7	7·8	4·3	4·0	5·6	4·3	5·0	4·4	5·3	5·8	5·4	5·3	5·3	3·9	74 o/o	6·0	6·0	12·0
RAVI	1·3	2·3	1·3	1·3	1·9	2·0	1·9	2·2	1·5	1·8	1·4	2·1	2·1	1·9	1·9	2·6	2·0	1·7	1·9	2·4	1·9	1·6	2·0	1·9	2·1	2·0	1·9	1·3	68 o/o	3·0		3·0
BEAS							3·5	4·5	2·8	3·4	3·8	4·7	4·0	2·6	2·5	5·6	3·9	3·5	5·5	3·8	3·1	3·5	3·8	3·7	3·9	3·2	3·2	2·6	73 o/o	5·7		5·7
SUTLUJ	3·3	5·2	3·3	3·6	2·8	3·4	3·3	4·5	3·2	3·2	3·6	4·1	4·0	3·5	3·6	4·4	3·4	3·4	3·8	3·6	2·6	3·7	4·1	4·2	3·8	4·0	3·8	2·6	68 o/o	5·0	16·0	21·0
JUMNA	2·2	4·4	2·5	2·3	3·1	3·3	2·9	3·7	2·4	2·5	2·5	4·4	2·8	2·6	2·1	3·7	2·2	2·3	2·3	2·4	2·8	2·3	2·4	3·7	3·8	3·9	3·8	2·1	55 o/o	4·4		4·4

Factors Affecting Stream Flow

(1) Evaporation.
(2) Absorption.
(3) Natural storage of underground waters.
(4) Vegetation and heavy forests will interpose an appreciable time element in the run-off. Forests receive a greater quantity of rain, snow, and heat than open land in the same vicinity, and mountain areas, whether bare or covered with forest growth, receive more rain than flat low lying country, that forests in mountain areas receive more rain than open land at the same elevation. The conservation of water supplies is inseparably connected with that of forests.
(5) Snow and ice may store up the precipitation for weeks or months.
(6) There may be unexpected drought or flood due to effect of abnormally dry or wet season.
(7) Natural storage features tending to equalize the stream flow as compared with the rainfalls.
(8) Lakes, ponds and swamps have great value in regulating the stream flow, and broad rivers have very frequently good storage possibility.

Gauging

Gauging should be taken at a place where the course is straight without bends, the flow is uniform, bridge piers or dams and pools and the bed should be permanent and there should be no boulder or weeds interfering with the gauging. Mark off with pegs any convenient length, say 200 feet of fairly uniform width. Find the average depth by soundings at intervals across the stream. Take at least three cross-sections one near each end of the selected reach and one near the middle, measure the depth at 10 or more points at equal distances across the width of each of these cross-sections.

Disposal of Rainfall :—A portion of the rainfall evaporates; a part is either absorbed by plant growth or by ground flow reaches the river or lake, and the third part flows into streams as surface flow or run-off.

Evaporation :—This is variable with time and is affected by the character of the soil and by atmospheric conditions such as the velocity and humidity of the wind velocity—temperature of the water itself and of the air varies inversely with the porosity of the soil and distribution of the summer rainfall. The nature of the earth's surface determines the rate at which moisture

is supplied. The depth of the pervious layer of soil is as important as the porosity of the soil; the depth of the water in the soil and its capillary action in bringing the water to the surface also naturally affect the evaporation. Evaporation from the soil surface is dependent upon the same factors which control evaporation from water surface, and, in addition, it is dependent on the degree of saturation of the soil surface.

Wind velocity generally tends to increase with elevation. With large surface area, the total evaporation will be slightly greater than the evaporation for a small surface area. Evaporation from flowing water is about 8 per cent. greater than from still water and an increase in water temperature produces practically corresponding increase in evaporation. A large amount of water is taken up by vegetation and evaporated while forests cause more evaporation than open fields. Thus the type and amount of vegetation and the state of its growth all affect the evaporation. It varies like rainfall in different localities at the same time and in the same locality at different times.

"In India it may be as much as 6 ft. in the year, but for details of many known reservoirs see Buckle'y Irrigation Pocket Book." (Page 8, Hydro-Electric Survey of India, Part I).

Flat slopes, lakes, swamps and all vegetation retard flow and promote seepage and provide greater evaporation opportunity unless the soil is quite pervious. They tend to promote regularity of stream flow but reduce the total run-off.

*The average evaporation from a free water surface is approximately as follows :—

Mean temperature, F.	30°	40°	50°	60°	70°	80°
Monthly evaporation (inches) ...	0·5	1·1	1·9	3·5	5·5	8·0

†Vermeule, as a result of an investigation of a number of catchment areas in the Eastern United States, gives the formulæ :—

$E = 15{\cdot}5 + 0{\cdot}16\ R$ for the whole year,

$E = 4{\cdot}2 + 0{\cdot}12\ R$ for the period December to May inclusive,

$E = 11{\cdot}3 + 0{\cdot}20\ R$ for the period June to November inclusive,

(Where E and R are the total evaporation and rainfall in these periods.)

* From Reports of U. S. Weather Bureau.

† Report of Geological Survey of New Jersy, 1904.

Dr. Walter Leather gives the following formula for estimating evaporation from water surfaces :—

$$E = 2 \cdot 0 (\log T - 1 \cdot 74) + 0 \cdot 33 (\log D - 1 \cdot 00) + 0 \cdot 36 (\log W - 0 \cdot 125)$$

where E = Evaporation in millimetres for 24 hours ; D = Dryness = 100 − (humidity at 8 A. M.) ; W = Mean wind velocity for the 24 hours in miles per hour ; T = Mean temperature in degrees F. for the 24 hours.

The relations of evaporation as established by Prof. Thomas Tate, which are of interest to water-power engineers, are as follows :—

(*a*) Other things being the same, the rate of evaporation is nearly proportional to the difference of the temperature indicated by the wet-bulb and dry-bulb thermometers.

(*b*) Other things being the same, the augmentation of evaporation due to air in motion is nearly proportional to the velocity of the wind.

(*c*) Other things being the same, the evaporation is nearly inversely proportional to the pressure of the atmosphere.

(*d*) The rate of evaporation from different substances mainly depends upon the roughness of, or inequalities on, their surfaces, the evaporation taking place most rapidly on the roughest or most uneven surfaces ; in fact, the best radiators are the best vaporizers of moisture.

For an approximate estimate of evaporation, Mr. Fitz Gerald's formula may be used. Calling—

V = the maximum force of vapour in inches of mercury corresponding to the temperature of the water ;

W = the velocity of the wind in miles per hour ;

E = the evaporation in inches of depth per hour ;

v = the force of vapour present in the air ;

Said formula has the form :—

$$E = \frac{(V - v)\left(1 + \frac{W}{2}\right)}{60}.$$

The force of vapour present in the air is, however, dependent on the temperature of the air and the height of the barometer, so that the value of v in the foregoing formula must be computed beforehand from the following relation :—

$$v = v_1 - \frac{0 \cdot 48 (t - t')}{689 - t'} h.$$

In which :—

v = the force of vapour in the air at the time of the observation ;

v_1 = the force of vapour in a saturated air at the temperature t′ ;

t = the temperature of the air in degrees centigrade, indicated by the dry thermometer ;

t′ = the temperature of evaporation given by the wet thermometer ;

h = the height of the barometer.

Pressure of Saturated Water Vapour, from 0° to 100° C., in mm. of Mercury.

Temp. C.	Pressure mm.	Temp C	Pressure mm.	Temp. C.	Pressure mm.
0	4·58	34	39·75	68	214·0
1	4·92	35	42·02	69	223·6
2	5·29	36	44·40	70	233·5
3	5·68	37	46·90	71	243·8
4	6·10	38	49·51	72	254·5
5	6·54	39	52·26	73	265·6
6	7·01	40	55·13	74	277·1
7	7·51	41	58·14	75	289·0
8	8·04	42	61·30	76	301·3
9	8·61	43	64·59	77	314·1
10	9·20	44	68·05	78	327·2
11	9·84	45	71·65	79	340·9
12	10·51	46	75·43	80	355·1
13	11·23	47	79·37	81	369·7
14	11·98	48	83·50	82	384·9
15	12·78	49	87·80	83	400·5
16	13·62	50	92·34	84	416·7
17	14·52	51	96·99	85	433·4
18	15·46	52	101·9	86	450·8
19	16·46	53	107·0	87	468·6
20	17·51	54	112·3	88	487·1
21	18·62	55	117·8	89	506·1
22	19·79	56	123·6	90	525·8
23	21·02	57	129·6	91	546·1
24	22·32	58	135·9	92	567·1
25	23·69	59	142·4	93	588·7
26	25·13	60	149·2	94	611·0
27	26·65	61	156·3	95	634·0
28	28·25	62	163·6	96	657·7
29	29·94	63	171·2	97	682·1
30	31·71	64	179·1	98	707·3
31	33·57	65	187·4	99	733·3
32	35·53	66	195·9	100	760·0
33	37·50	67	204·8		

Absorption :—A very large part of the precipitation is absorbed by the ground. The quantity varies greatly on the rate of precipitation, nature of the soil, slope of the drainage surface, temperature and vegetation. It is this ground supply which enables stream flow to be maintained during protected periods of slight rainfall and areas of little or no underground flow are subject to violent droughts and extreme floods.

Approximate quantity of water absorbed by soils.

Material.		Volume of water absorbed per 100 of material.
Sand	...	30 to 40.
Coarse sand	...	39·5.
Chalk soil	...	49·5.
Sandy soil	...	49·5.
Clay	...	50·53.
Loam	...	45 to 60.
Peat sub-soil	...	84.

Run-off

The percentage of the total fall which reaches the stream at any point is known as the "Run-off." It varies from 80 to 90%.

It is that part of the rainfall which remains after nature's need of moisture has been supplied in the form of evaporation and absorption.

The run-off depends upon the geology, topography, the size of the catchment area, the temperature, the vegetation by its effect or evaporation and distribution of flow, the distribution of the rainfall and type of the rainfall. The temperature particularly equalizes the flow in rivers fed by glaciers or snow fields.

Rainfall is for the most part fairly distributed in the temperate zone throughout the year. In the northern part of India, *e.g.*, up the Punjab, etc., it is confined to a monsoon period with perhaps eight or nine months practically dry to follow. Where there are high mountains, much of the water is precipitated in the foam of snow which serves as storage reservoirs of large dimensions and tends to equalize the flow throughout the year. Such is the case in the streams up the Himalayas, *e.g.*, in the Nepal Territory.

The north-east monsoon is sometimes a complete failure, but the south-west monsoon is regular and dependable and the

Mettur reservoir will store the excess flow during the south-west monsoon and distribute it evenly during the succeeding period.

It may be mentioned that the Mysore Government have a dam across the Cauvery at Kannambadi about 155 miles upstream and the question of the amount of water the Mysore Government had a right to store, was one of vital importance to a reservoir below.

The total estimated cost of the Mettur project which includes the canal system as well as the headworks is Rs. 7,37,08,000.

High falls are often found in mountainous countries where the rivers have a great catchment area extending over high table lands and terminating in glaciers fed by perpetual snows. Such falls may be utilized for big hydro-electric schemes with great advantage; they are very precious stores of energy provided there are sufficient industries to absorb the energy with profit.

Although such mountainous countries have natural advantages, sufficient falls can be found in nature or may be artificially developed, as great rivers in flat countries can be utilized for very big schemes. The falls of the Ganges canal such as at Bahadarabad are purely artificial falls which have been utilized to supply power and light to a large number of cities in United Provinces.

Vermeule's equation for annual run-off is—

$$S = R - (11 + 0{\cdot}29\ R)\ M\ ;$$

where S = total annual run-off in inches ;
R = total annual rainfall in inches ;
M = a factor which depends upon the mean annual temperature.

This equation is the result of studies to determine the effect of forests on evaporation and, while extreme accuracy is not claimed for it, the equation has received wide publicity.

Short period maximum or flood run-off :—In designing headworks, spill-ways, dams and waste weirs or escapes, remember the effect of a sudden phenomenal but short-lived flood in a steep catchment. A very heavy flood lasting for a short time may do heavy damage but cannot be of much voltage for power. In calculating such floods Colonel Dicken's formula is very generally used in India.

$$D = CM^{\frac{3}{4}}\ ;$$

where D = maximum discharge of river in cusecs ;
C = a coefficient ;
M = area drained by the river in sq. miles ;

Dicken's value of C was 825 for annual rainfall of about 36 inches, but he held it applicable to falls from 24 to 50 inches. "The formula takes no cognizance of the size or shape of the catchment, nor of its declivity, and provides no factor taking account of the variations in the rainfall"— (Irrigation Pocket-Book, Third Edition, page 299),

Dicken's formula is probably more particularly suited to the conditions of Bengal and other parts of Northern India, but it may be used in other countries.

Ryve's formula :—

$$Q = CM^{\frac{2}{3}};$$

where Q = discharge in cubic feet per second ;
M = area in square miles ;
C = a coefficient usually ranging from 450 to 1000 according to the nature and situation of the catchment. Rainfall data are also taken into consideration by comparison with established results.

This formula is largely used in Southern India.

For a considerable range of the catchment areas the coefficient 1000 is a good provisional figure. It applies to many parts of the tropics in hilly country. For small catchments in such districts the discharge may correspond to higher values up to 2000 for the maximum in an indefinite period of time. But the site of the Gatun dam, catchment 168 sq. miles, the maximum flood estimated from records, corresponded to a coefficient of 1525.

Long period aggregate run-off :—In order to be of any use to us the actual quantity of water must pass a particular point— either in its natural course or due to main control. If there is a storage reservoir of sufficient size for a year's rainfall, the outflow or draft can be exactly controlled and the actual run-off is the criterion. In upper India there is only one monsoon ; consequently about 8 months even big rivers almost dry up. In order to get continuous power throughout the year sufficient water must be stored up from previous monsoon and sometimes this is necessary to supplement the flow during successive years of poor monsoon rainfall. Hence, it is extremely important to determine the total run-off from a given catchment during the whole monsoon period and also in the case of casual heavy storms at other items.

The average of annual flow for India excluding Burma, Eastern Bengal and Assam is given as 41·6 per cent.—although

this is not very reliable. "The average total rainfall on these tracts in India averages 37½ inch. The Indian Irrigation Commission estimates that this depth is disposed of as follows :—

22 inch or 59 per cent. is absorbed (1) in sustaining plant life, (2) in maintaining the moisture of the soil, (3) in replenishing the subsoil water supply, and (4) by evaporation.

2·25 inch or 6 per cent. is utilized in artificial irrigation.

13·25 inch or 35 per cent. is carried away by the rivers to the sea."

Barlow, the great authority on Indian Irrigation, classifies catchments as follows :—

A = flat cultivated and black cotton soil catchment.

B = flat partly cultivated and stiff soils.

Rate of flow small ; absorption and loss large ; run-off small.

C = average catchment.

D = hills and plains with little cultivation.

Rate of flow large ; absorption and loss small ; run-off high.

E = very hilly, steep and rocky with very little cultivation.

His classification of rainfall into light, medium and heavy daily rainfall involves the entire omission of very low rates of discontinuous rainfall, as follows :—

Light falls under ½ inch in 24 hours should be entirely omitted unless continuous for several days ; falls from ½ inch to 1 inch in 24 hours should be omitted if there has been no fall before or after. Falls from 1 inch to 1½ inch 24 hours, when not followed or preceded by similar or greater amounts, are considered light.

Medium falls are those from 1 inch to 1½ inch in 24 hours when followed or preceded by any but light falls ; also all falls from 1½ inch to 3 inch per diem.

Heavy falls are those over 3 inch or continuous falls at a rate of over 2 inch a day ; also all falls of an intensity of 2 inch an hour or over.

Barlow then assumes the following percentage run-off :—

Type of fall.	Type of catchment.	A.	B.	C (Average).	D.	E.
Light falls	Per cent.	1	3	5	10	15
Medium falls	Per cent.	10	15	20	25	33
Heavy falls	Per cent.	20	33	40	55	70

This is also emperical and is defective in many ways.

Mr. J. W. Meares has modified Barlow's coefficients as follows :—

Catchment.	A.	B (3 to 7 sq.mi.)	C. (10 sq mi)	D. (78 sq. mi.)	E.	Average.
Light falls ...	...	(43)	28	(10)	...	27
Medium falls ...	...	36	51	39	...	42
Heavy falls ...	...	48	55	83	...	62
Mean of percentages	...	42	44	43	...	...

A knowledge of the 'run-off' for all seasons over a prolonged period of several years is of prime importance to the designer, and is in fact his starting point.

Rainfall and Run-off at Pykara :—There are several rain gauges in different areas of the whole Pykara catchment and an examination of the seasonal rainfalls at the various stations shows that the Pykara gauge proper indicates the average conditions over the catchment more closely than any of the others. This gauge has, therefore, been taken as showing the proportionate annual variation on the whole catchment. The catchment receives both the South-West and the North-East monsoons, but the run-off due to the South-West monsoons is very much greater than that due to the North-East. For purposes of hydrometric survey the year has, then, been divided into three seasons.

Method now used for calculating flow :—To deduce the monthly flow rainfall and run-off statistics from other catchments in which the most similar conditions prevail have been taken and the results applied to the computed rainfall on the Pykara catchment. The two catchments chosen are "Thalipuzha" and "Sandy Nalla" basises and have similar natural features to that of "Pykara" lie on each side of the Pykara catchment.

Thalipuzha :—This catchment is on the edge of the Western Ghats in the Wynads, and is subject *to a very heavy rainfall* of which about 88% occurs during the S.-W. monsoon. It is a small flat valley surrounded by high steep hills covered with thick grass and heavy jungle. The elevation varies between 2,800 and 6,000 ft. m. s. l. and the catchment area is 6·7 sq. miles.

Sandy Nalla :—This catchment adjoins the Pykara and is similar to it as regards elevation and ground. The rainfall, however, is *considerably lower* and is affected only to a comparatively *small extents by the S.-W. monsoon.*

Rainfall and run-off records of these two catchments—one of which corresponds to the S.-W. monsoon conditions of Pykara and the other to the N.-E. and dry months—are available since 1925. For purposes of our conclusion, it can be roughly seen that Thalipuzha, the area of heavy rains, has an average rainfall of about 350″ and run-off of about 300″, the difference between the two being for all years near about 50″. Sandy Nalla has roughly a rainfall record of nearly 50″ and a run-off of over 30″ keeping up a difference fluctuating near about 20″. *The conclusion then is that the relationship between rainfall and run-off is a rough difference for each catchment and that the difference is greater for catchments with greater rainfalls.* This is exactly where our doubtful premises begin and the above conclusion, though perhaps apparently reasonable and justifiable, is still only "probable" and only the end—a protracted, prolonged end—can justify the means. And there is time for that too.

So, these two catchments have been taken as guides in deducing the run-off at Pykara as the Pykara basin rainfall *is intermediate* between those of these areas and the difference of rainfall and run-off on the Pykara is, therefore, tacitly expected to be some figure betwixt and between those of the two catchments. The Pykara basin rainfall varies between 180″ maximum with an 82% S.-W. monsoon and a minimum of 60″ with only a 34% S.-W. monsoon. These two extreme conditions approximate to those of Thalipuzha and Sandy Nalla respectively and it seems reasonable to assume that the total difference between rainfall and run-off in the wetest year will be considerably greater than that of the driest and that intermediate rainfalls will have proportionately intermediate results. This assumption of the difference varying with the rainfall in the same catchment is justified by the data available for several monsoons.

Curves giving the difference between rainfall and run-off for maximum and average years in the Pykara basin *are available.* It is easy to see how they have been drawn from the above data. The "differences" for the two catchments can be plotted cumilatively and to scale drawn, showing the proportionate intermediate values of the losses for varying the total rainfalls each month. Thus deduced, the rainfall, run-off % vary between a maximum of 81 % and a minimum of 63%. These percentages have been considered as satisfactory and not unreasonable for this catchment.

Readings of the gauges in the other stations in the catchment give results which fairly correspond with that given above.

Run-off :—The river flow records are available for the whole of the three years 1925, 1926 and 1927, which were used with some changes for the design purposes. There is rectangular notched weir across the Pykara river and an automatic water stage recorder. The run-off as calculated from this is given below.

	1925		1926		1927	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.
January, February & March	0·8	3·9	4·0	4·6	0·6	1·7
April and May ...	14·8	2·2	7·7	1·8	6·7	0·6
June	14·6	23·1	5·4	2·4	25·0	32·2
July	23·6	31·5	34·5	43·4	38·9	52·1
August	21·6	21·0	27·8	44·0	10·6	17·7
September	9·8	9·0	11·8	11·0	7·2	6·9
October, Nov & December...	14·4	12·7	8·4	8·1	7·1	7·4
Total ...	99·6	103·4	99·6	115·3	96·1	118·6

This gives results of run-off in excess of rainfall especially in the South-West monsoon months. The rainfall of a catchment may be less than the run-off if the area of the catchment has been incorrectly surveyed or if subterranean waters, having their source outside the catchment, come to the surface within the basin. Both of these were found to be impossible and the only possibility is the incorrect measurement of the run-off. Examination showed that the ratio of the surface to mean velocities in many cases varied from about 0·46 to 0·99 for observations taken at approximately the same stages. These variations were specially so in the South-West monsoon and might be due to the strong South-West wind blowing down the river at this season tending to increase the surface velocity.

Further investigation was carried on in this respect and rainfall and run-off readings taken of Thalipuzha and Sandy Nalla basins in which conditions in respect of elevation and ground approximate those of Pykara. Further the maximum and minimum year occurred here as also in Pykara at the same time with corresponding seasonal falls. Since these extreme conditions approximated to those of Thalipuzha and Sandy Nalla, it seemed reasonable to assume that the percentage of run-off also would be the same. Also the difference between rainfall and run-off in the wetest year is considerably greater than that of the driest, and intermediate rainfalls will have proportionately intermediate differences. The rainfall run-off percentages vary between a

maximum of 81 and a minimum of 63. The computed monthly rainfall and run-off in inches for Pykara basin for a period of 45 years is available.

The above is a complete analysis that has been done, and further work in this direction is going on. These results of the run-offs of the Pykara catchment during the various seasons are expected to approximate actual conditions.

Long before it becomes necessary to consider seriously the utilization of the greater part of the run-off of a minimum year during the S.-W. monsoon, reliable information about the flow data may be available. However, for the preliminary estimates and calculations, the method detailed above has been followed. The driest year since 1882 was 1918. But, however, the years were divided into monsoon periods and dry periods and the driest for each period noted over these years on record. 1918 had the lowest monsoon and '26-'27 the lowest summer. An arbitrary minimum year combining these two extreme conditions together—which is highly improbable and so very safe—was chosen for the estimates and preliminary investigations.

Taking the question of storage to act as regulating reservoirs, it is absolutely necessary to have a careful consideration. The topography embraces a number of feasible storage sites and to take full advantage of this scheme, storages should be developed to suit the varying load conditions. In many projects such methods are not possible, but fortunately this catchment is admirably suited for such a development. The ground is in general not suitable for high dams owing to the high cost of foundations. There is, however, bed rock at all possible dam-sites and structures up to 100 ft. can be constructed at a reasonable cost.

The most economically feasible sites for storage are:—

		Run-off of Catchment.		Approximate ratio of unit cost.
		Min. year. M. C. Ft.	90 % year. M. C. Ft.	
1.	Mukurti	1080	1614	15
2.	Pykara	1447	2033	27
3.	Porthymund	459	581	35
4.	Sandy Nalla	627	848	43
5.	Porson's Valley	396	517	50
6.	Glen Morgan	66	84	81

The question of capacities of the various reservoirs is important. A site may be developed to store the minimum annual run-off of its catchment or an amount greater or less than this figure. When there are several reservoir sites and all may be needed some time later careful study was necessary as to the best method of

development. An isolated power installation can obviously be developed only up to the run-off of the minimum year, if no storage or carry-over is contemplated. In such cases, the load in the earlier stages of development may not warrant the utilization of the whole of this run-off, so that only sufficient storage is required to tide over the worst part of the year, but minimum seasonal water condition may not occur in the minimum year; it may happen in a year with a higher run-off. Hence sufficient storage must be provided for.

The load forecast and initial demand, as will be shown later, do not demand the utilization of all the reservoirs. The only reservoir necessary in the first few years of operation is to supplement the flow of the worst portion of the year. The development of Glen Morgan and the forebay were decided solely on practical operating conditions.

A limited storage was found to be necessary before the regular operating period commences, to provide water supply to camp and constructional purposes. A small amount of storage is also needed to ensure a continuous flow for the auxiliary plant during dry months. Low initial investment rather than low unit cost governs the decision in this case. *Glen Morgan is the site chosen because it is necessary as an alternative water supply for the cooling of transformers and bearings at the power house,* and can be used as a regulating basin when Sandy Nalla reservoir is constructed. No other site fulfilled these conditions so well and because of the high unit cost. The reservoir is built with a capacity of 26 m. c. ft. much below the minimum run-off of the catchment and the dam is 45 ft. high. A regulating forebay is constructed by the construction of two bunds, the upper one being 39 ft. and the lower one 62 feet having a capacity of 65·63 m. c. ft., of which 58·8 is effective. Then reservoirs will be constructed when the load warrant it.

The table of run-off as computed above shows that during the months of January, February and March—the dry season—run-off is more or less steady gradually diminishing from January to March. This, it is easily seen, is in accordance with what one would expect its magnitude depending on the previous monsoon conditions and not on the rainfall in the dry months. The method adopted in computing the run-off during these months does not allow for exceptional conditions and therefore gives higher rates than are likely to have occurred during dry months when heavy rainfall and late monsoon occurred. There are, it seems, only a few instances of these exceptional conditions since 1882 and these do not occur in the minimum years which alone will

be taken for design purposes. The monthly discharges thus estimated with a series of emphatical assumptions form the basis of all calculations of the designer.

If the above figures are too conservative, the regulated run-off will be higher and some money would have been spent on superfluous storage; if, on the other hand, the deduced figures are too high, more storage will have to be provided for than contemplated.

CHAPTER V

MEASUREMENTS OF WATER-POWER

Available Water-Power in India.—The flow of a stream is a greatly variable quantity, varying from day to day, month to month, and season to season, and is different for the same part of the season in any one year than in any other year, and much more so in India where the rainfall is seasonal and comes down chiefly during the monsoon period from May to August followed by eight comparatively dry months. In the case of the snow-fed rivers this natural deficiency in summer season is made up to a great extent by the melting of snow, which, if excessive on any occasion, does actually give rise to floods in some cases in the hot season. Again the country being so vast, even the monsoon fall differs very much in different parts and is thus more certain in some parts than in others which are consequently subject to protracted periods of droughts for years if the rainfall fails.

In any hydro-electric scheme, therefore, if power is to be developed continuously, and the conditions are such that there are no storage possibilities available or even if available are too costly, the plant capacity to be installed must depend upon the minimum flow of the river. Now whether this minimum is to be the absolute minimum of the stream, or the minimum of an average year of neither very high nor very low rainfall, or again the average of the minimum flow of all the years for which records are available is a point for careful consideration. Usually in practice the first of these is not economically profitable, and is seldom acted upon. The method recommended for arriving at the quantity of water available which can be relied upon in such cases is as follows :—Examine carefully the records of flow available extending over as long a period as possible. Make up the missing record for any month in any year from the records for that month of any other similarly situated stream. Arrange the months of the year according to the days of lowest flow in each. Take the lowest of the six high months as the basic month of the year. Find the average of the two lowest seven-day consecutive period flows. The average of all such averages for all the years of record is the amount of flow to be adopted in the calculations.

Under Indian conditions generally the values to be adopted may be taken as two-thirds of the above except in Bombay and Central Provinces which depend to a very great extent on monsoon storage for their supply and in their case the figure taken should be $\frac{1}{3}$ instead of $\frac{2}{3}$.

The maximum plant capacity that can profitably be installed under no storage conditions in India may generally be taken 3 times the minimum continuous power determined as above for most parts, the corresponding figures for Bombay and Central Provinces being $1\frac{3}{4}$ and $1\frac{1}{4}$ respectively.

Serial No.	Place.	Owner.	Source of water.	Capacity of plant 1933.	Minimum perennial flow of source.	Storage in million ft.	Gross head ft.	Length of channel miles.	Capacity of channel cusecs.	Length of pipe line.	Cost of hydraulic development (Rs. per kW)
1	Jhelum ...	Kashmir State.	Rapids in Jhelum river.	...	4,000	*Nil*	395	6·5	500	750	565
2	Jammu ...	Do.	Ranbir Canal ...	..	500	*Nil*	26	*Nil*	...	...	317
3	Khopoli ..	Tata Company.	Monsoon rain stored.	...	*Nil*	10,000	1.725	4·6	480	12.514	400
4	Andhra Valley	Do.	Do.	...	*Nil*	12,000	1,743	*Nil*	...	4.700	...
5	Gokak falls ...	Private ..	Ghataprabha River.	...	136	907	210	½	200	275	...
6	Cauvery falls ...	Mysore State	Cauvery River	50,000 H. P.	1,000	500	415	3¼	1,000	1.000	550
7	Mussoorie ...	U. P. ...	Bhata Stream	...	8½	*Nil*	1.000	*Nil*	...	4.350	240
8	Darjeeling ...	Municipality	Tributaries of Ranjit.	...	10	*Nil*	275	2·9	? 100	720 1.450	120
9	Bhatgar ...	P. W. D ...	Yelwandi River & Whiting lake	...	1·50	5,312	46 to 101	*Nil*	...	...	...
10	Simla ...	Municipality	Nanti Khad Stream.	...	19	*Nil*	540	2¼	32	1.300	710
11	Amritsar ...	P. W. D. ...	Bari Doab Canal	...	1.050 for 240 days.	*Nil*	6 to 9 7	Short diversion.	2,300	*Nil*	630
12	Patiala ...	Patiala State	Sirhind Canal Ghaghar branch.	...	3,000	*Nil*	8	·2	550	*Nil*	900
13	Ganges head works.	P. W. D.	Ganges Canal falls.	...	·3	*Nil*	19	Short diversion.	660	*Nil*	300
14	Cordite Factory Nilgiris.	Government	Hill Streams ...	...	..	2 lakes 9 & 12	650	*Nil*	*Nil*	2,229	368
15	Khatmandu ...	Nepal State	Satmoday Spring	...	...	...	678·5	...	*Nil*	2.805 3.806	...
16	Malakand ...	N. W. F. P.	Swat Canal ...		..	...	30	2½	125	45	630

Summary of probable minimum continuous water-power in India

Province or state.	Number of sites developed.	Water-power now developed (site capacity)— kW. continuous.	Plants installed kWs.	Areas investigated but not developed— kW. continuous.	Areas and sites not investigated Probable kW.	Probable total kWs.	Probable ordinary minimum power.	Probable power for maximum development
1	2	3	4	5	6	7	8	9
							kW.	kW.
Assam ..	...	...	...	109,000	300,000	414,000	621,000	1,200,000
Baroda ...	...	...	...	4,000		4,000	...	...
Bengal ...	2	600	600	14,250	650,000	669,850	1,000,000	1,200,000
Bihar & Orissa ...	...	...	...	12,550	30,000	62,550	95,000	150,000
Bombay ...	4	187,900	187,970	272,560	20,000	644,310	773,000	1,000,000
Burma ...	3	3,370	6790	155,800	300,000	951,570	1,327,000	3,000,000
Central India ...	...	...	...	280	...	680		..
C. P. & Berar ...	...	...	...	13,700	10,000	137,560	165,000	180,000
Cochin ...	...	...	...	4,000	...	4,000	...	...
Coorg ...	...	...	.	...	...	1,500	...	...
Gwalior ...	...	...	...	42,300	...	43,300	...	...
Jammu & Kashmir	2	105,830	4,670	...	20,000	305,330	458,000	650,000
		400	...	...	...	400		
Madras ...	3	740	...	32,670	5,000	92,310	138,000	300,000
Mysore ...	1	24,000	..	20,000	...	48,500	...	...
N. W. F. Province	1	(250)	250	...	(1,000,000)	(1,000,000)	(1,000,000)	(1,000,000)
Patiala ...	1	450	450	...	...	450	...	..
Punjab & Canals .	3	1,880	...	129,270	662,000	793,150	1,190,000	2,400,000
Amristar ...	...	525	..	...	...	...		...
Rajputana ...	..	...	2,945	...	.	160	...	
Sikkim ...	...	...	...	5,000	...	5,000	...	...
Tavancore Pulivasal, Munoar.	1	450	...	...	...	450	...	...
U. P. & Canals ...	2	5,770	6545	378,900	20,000	403,370	605,000	1,000,000
		1,775						
		6,545						
Other areas ...	...	...	...	...	..	...	160,000	300,000
Total ...	22	213,140	...	1,194,280	3,017,000	5,582,000	7,532,000	12,680,000

Watts per capita in various countries.

		Watts per capita.
Canada	...	148
Australasia	...	62
South Africa	...	57
British Isles	...	33 = 1 ordinary glow lamp.
India	...	1 or less.

If population is a poor criterion, the area per kilowatt ($1\frac{1}{3}$ horse-power) installed may be taken.

Area per kilowatt in various countries.

British Isles	...	1/12 sq. mile per kilowatt.
South Africa	...	1·4 do.
Canada	...	3·5 sq. miles do.
India	...	7·6 do.
Australasia	...	1·1 do. Largely uninhabited.

Summary of known power in use in India.

		B. H. P.	
Assam	...	22,550	
Bengal	...	25,318	exclusive of Calcutta area.
Calcutta area	...	176,200	
Bihar	...	2,325	apart from collieries, etc.
Bombay Presidency	...	32,872	
Bombay City area	...	750,000	
Burma	...	17,750	exclusive of rice mills, etc.
Central Provinces	...	32,773	
Madras	...	60,000	
Punjab	...	90,000	
United Provinces	...	38,548	
Grand Total	...	1,248,336	or 936,000 kWs.

For details *vide* Hydro-Electric Survey of India, Vols. I, II and III.

For a given quantity of the gross horse-power the value should ordinarily commence and finish at the diversion or head-works intake to the developed waterways, and the horse-power value to decrease in amount all along the system of waterways, etc., up to the electric distribution or the consumer's service. In estimating available power, use can be made of the following constants.

Efficiency constant for horse-power and kW. to be expected at definite points in the hydro-electric system is given as follows :—

*Table.—Showing the Efficiency Constant for h. p. and kW. To be expected at difinite points in the Hydro-Electric System.

Conditions.	System Power Constants			
	H. P.		kW.	
(a) At the headworks or intake	HQ	0·1134	HQ	0·0846
(á) Gross or theoretical power	,,	0·1134	,,	0·0846
(b) At the turbine shaft	,,	0·0907	,,	0·0677
(c) At the generator switchboard	,,	0·0855	,,	0·0636
(d) At the end of the transmission line ..	,,	0·0796	,,	0·0586
(e) At the receiving-station switchboard ...	,,	0·0760	,,	0·0563
(f) At the consumer's terminals	,,	0·0700	,,	0·0518
(g) At the 1. p. distribution service ...	,,	0·0630	,,	0·0466

Where Q = discharge ft.^{3}/sec., and H = ft. head.

To (g) this would equal to an over-all efficiency of nearly 50 per cent. Hence a good ordinary value to assume for estimating purposes would be around 60 per cent. over-all efficiency up to the bulk power consumer or 75 per cent. to switchboard.

*For the Large Hydro-Electric Systems, the Estimated and Probable Over-all Losses in Per Cent Values

Hydro-Electric System Losses.	Per Cent		Values.
	Best Modern.		Poor installation.
	(a)	(b)	
1. In the developed waterways, including headworks, short conduit and pipe line	2	4	11
2. Turbines (main units, exciter units, etc.)	7	10	18
3. Generators and exciters, etc. ...	2	3	5·5
4. Step-up transformers	0·8	1·5	2·2
5. Transmission lines	6	8	15
6. Step-down transformers	0·8	1·5	2·2
7. Synchronous machines, substations, and distribution centres	4	8	12
8. Feeder circuits	5	10	16
9. Low-pressure transformers and distribution service	5	9	18
Total ...	32·6	55	99·9

* The best over-all efficiency of all the developed waterways from the headworks intake to the turbine shaft (including such rare cases as the late Niagara installation) is :—

Best modern—(*a*) 91 per cent.
Mean value—88·5 per cent.

As bulk supply will usually include those items between No. 1 and No. 6 inclusive, the very best over-all efficiency will be :—

Best modern—(*a*) 81·4 per cent. Poor future—...per cent.
(*b*) 72·0 ,,
Best mean over-all efficiency—76·7 per cent. (modern).
,, ,, ,, 81·3 ,, (future).

Measurement of Water-Power :—The power derived from the fall is proportional to the product of the weight of water and the vertical height of the fall. The rate of flow or the value of a given total quantity of water in terms of power is directly proportional to the working head which is the difference in level in feet between the forebay or pipe entrance or head waters as the case may be and the point where the water finally ceases to do work, diminished by the losses in the passage except turbine losses which are considered in determining the turbine efficiency. The water horse-power is being converted to mechanical horse-power when it is reduced by a certain percentage which depends upon the efficiency of conversion and this mechanical power appears at the wheel shaft.

The measurement of water should be accurate and should be with particular reference to—

(1) Minimum quantity of water in dry seasons.
(2) Average quantity of water during the whole year.
(3) The maximum quantity of water for which the water ways and hydraulic works must be provided.

Efficiency of Hydro-Electric Plant

† " At the most economical load the efficiency of various types of hydraulic turbine or wheel varies from 85 to 90 per cent. in large sizes, while the corresponding over-all efficiency of alternators, including excitation losses, will be from 95 to 97 per cent. Thus the over-all efficiency under *the most favourable conditions* will vary from 81 per cent. to 87 per cent. In practical working,

* Pages 91 and 156, Practical Water-Power Engineering, by W. T. Taylor.
† The General Principles of Development and Storage of Water for Electrical Purposes, fro the Journal of the Institution of Electrical Engineers, Vol. 57, No. 283, June, 1919,

however, the most favourable conditions are not maintained; the plant will not all be running at full load and there will be some appreciable additional loss in all systems having any length of pipe—a loss increasing with time and incrustation. In low-head plants eddies also affect the efficiency. For practical industrial purposes it will be sufficient to assume the over-all commercial efficiency, E.

TABLE.

Over-all Commercial Efficiency E.

For	500 kW.	... ...	74 per cent.
,,	1000 ,,	... ...	76 ,,
,,	1500 ,,	... ...	78 ,,
,,	2000 ,,	... ...	80 ,,
,,	3000 ,, and over ...	...	82 ,,

Using the convenient irrigation unit of a "cusec" for a flow of a cubic foot of water per second, we then have :—

$$\text{E. H. P. output} = \frac{\text{cusecs} \times 62{\cdot}4 \times \text{gross head in ft.}}{550} \times \frac{E}{100}$$

If H^1 = total fall of level from the point where the water is taken from the stream or river to the point where it discharged into it again. A part of this fall is lost in overcoming friction of the various parts of the water way. Let this part lost by friction be h. The gross available work of the fall = GQH = 62·4QH.

Neglecting losses from friction and leakage the power of the fall may be utilized in various ways :—

(1) The weight in lbs. (62·4 Q) of water may be placed directly on the turbine and arranged to descend in contact with it through the distance H feet. This is by *gravity.*

(2) The water may descend in a closed conduit, making the pressure in lbs. per square inch at the bottom of the conduit equal to pQ = 62·4 HQ. This is by *pressure.*

(3) The water may be allowed to acquire velocity $V = \sqrt{2gH}$ $= 8{\cdot}025\sqrt{H}$. The kinetic energy of Q cubic feet will then be $0{\cdot}5\, GQ\,(V^2/g) = GHQ$. This is by *inertia.*

($H = V^2/2g = 0{\cdot}0155\, V^2$); where $g = 32{\cdot}2$.

If Q cusecs of water act by weight through a distance h′ ft. at a pressure p due to h′ of fall with a velocity V due to h″ of fall so that $h' + h'' + h''' = H$ (*vide* Burnell's theorem); then apart from the energy wasted by friction or leakage, etc., work done will be $GQH + pQ + (G/2)\,Q\,(V^2/g)$ foot lbs. the same as if the

water acted simply by its weight while descending H feet. If the net or effective head H is given in feet and the quantity of water in cusecs, then, for the purpose of preliminary rough calculation, we may assume the efficiency e to be 75 per cent.

Therefore, the amount of power derived from a given fall of water = HP = QH 0·085. If the effective head is in feet and the quantity in gallons per minute HP = QH 0·00023.

The available gross water horse-power is—

$$H\,P = \frac{Q \times 62{\cdot}4 \times \text{net head}}{550} = QH\ 0{\cdot}1134545 + \cdots\cdots$$

Brake horse-power is the power delivered at the turbine shaft and taken from the turbine shaft ; efficiency of 75 per cent. should be taken.

*Water cannot perform its work in turbine without certain loss of energy. The main hydraulic sources of loss are :—

(1) Loss by fluid friction impact and eddies.
(2) Leakage between guide apparatus and wheel and casing.
(3) Residual velocity in tail water.
(4) Loss due to pressure change and speed regulation. The total losses may amount from 9 to 18 per cent. according to the size of the turbine. Allowing for bearing friction, the over-all efficiency will generally be between 77 to 89 per cent. according to size and circumstances. As, however, we are more concerned in kilowatts than with electrical horse-power, it will be convenient to use a direct conversion factor avoiding decimals of no real significance, as follows :—

For	500 kW.		=	cusecs	×	gross	head	×	0·062
,,	1000 ,,		=	,,	×	,,	,,	×	0·064
,,	1500 ,,		=	,,	×	,,	,,	×	0·066
,,	2000 ,,		=	,,	×	,,	,,	×	0·068
,,	3000 ,,	or over	=	,,	×	,,	,,	×	0·070

"For rough purposes indeed it is quite sufficient to take kilowatts = cusecs × feet/15, or B. H. P. of turbine = cusecs × feet/11 for mechanical power as reliable mean figures, except for small installations where the divisor will be higher. Thus—

6500	cusecs on	3·5 ft. head	will	give	1500 kW.
650	,,	35	,,	,,	,,
65	,,	350	,,	,,	,,
6·5	,,	3500	,,	,,	,,

* Hydro-Electric Survey of India, Vol. II, by F. E. Bull and J. W. Meares.

The power obtainable from any source of waterfall is approximately as follows :—

Kilowatts = Flow in cusecs × head in ft. /15.

Total energy available from a source whether from flow or stored water is found by—

Kilowatt-hour = thousands of cubic ft. of water × head in ft./56,

where large storages are to be determined and the unit is in kilowatt year, *i.e.*, 365 × 24 or 8760 kWh.

KW. years = millions of cubic ft. of water × head in ft./500.

Useful Hydraulic Data (in round numbers generally).

" Flow

1 cusec flow = 1 cubic foot per second = 62·4 lbs. per second,
= 60 cubic feet per minute = 3,744 lbs. per minute,
= 374 gallons per minute,
= 3,600 cubic feet per hour,
= 86,400 cubic feet per day,
= 2·6 million cubic feet per month,
= $31\frac{1}{2}$ million cubic feet per year.

Gallons per minute = cusecs × 374.

$$\text{Cusecs} = \frac{\text{Gallons per minute}}{374}.$$

"Regulating Storage

100,000 cubic feet stored will give 1·16 cusecs for 24 hours.
" " " " " $2\frac{1}{4}$ " " 12 "

" Main Storage

$31\frac{1}{2}$ million cubic feet stored is equivalent to 1 cusec for a year; but owing to evaporation and percolation more storage would actually be required. In round figures :–

1000 million cubic ft. stored will give 30 cusecs continuously for a year.

" Reservoirs

1 acre = 43,560 sq. ft.
1 acre of water 1 ft. deep = 43,560 cubic ft.
= 1 acre foot.
1 sq. mile = 640 acres = 27·88 million square feet.
1 square mile of water 1 foot deep = 1 square mile foot, = 27·88 million cubic feet, which will (in practice) give a flow of about $\frac{3}{4}$ cusec for 12 months, $2\frac{1}{4}$ cusecs per 4 months or 3 cusecs for 3 months.

"Catchments

1 inch of rain = 100 tons per acre or = 3630 cubic feet per acre.
= 22633 gallons per acre.
= 1 cusec per acre per hour.
= 64,000 tons or 23·23200 or 2·33 millions cu. ft. per square mile.
= 14,485,000 gallons per sq. mile.

Under the like conditions rain falling at the rate of one inch an hour is equivalent to 1 cusec per acre per hour or 640 cusecs per square mile per hour.

In practice, however, only a percentage, and often a small percentage, will reach the reservoir.

"Power from Flow

On the practical basis assumed throughout this report

$$\frac{\text{Cubic feet flow} \times \text{head in feet}}{11} = \text{Electrical horse-power.}$$

If 15 is the divisor, the result is in kilowatts.

On small projects 12 and 16 may be used.

"Power from Storage

(*i*) Regulating : $\frac{\text{Thousand cubic ft. stored} \times \text{head in ft.}}{42}$

= E. h. p. hours.

If the divisor is 56, the result is in kW. hours or B. O. T. units.

(*ii*) Main. $\frac{\text{Millions cubic ft. stored} \times \text{head in ft.}}{370}$

= E. h. p. years.

If the divisor is 500, the result is in kW. years.

Millions cubic feet × head in feet × 17 = B. O. T. units.

Small falls from 3 feet up to 5000 feet or so is capable of being utilized :—

NOTE.—1 h. p. hour = 0·746 kW.-hour or B. O. T. unit of electricity.
= 1·980000 ft. lbs. = 2580 B. Th. U.
= $2\frac{1}{4}$ lbs. of water evaporated at 212° F.

1 kW.-hour or
B. O. T. unit = 1000 watt-hours.
= 1·34 h. p. hours.
= 2656400 ft. lbs.
= 3440 B. Th. U.
= 3lbs. or 0·3 gallons of water evaporated at 212° F.

A horse-power year or a kW.-year is 8760 times a h. p.-hour or kW.-hour.

*Available Power from Rainfall

In terms of average values of rainfall, catchment area, and head, the available power can be expressed.

H.P. = 0·00828 f. m. h.; E. H. P. = 0·0073 f. m. h.; kW. = 0·0062 f.m.h.—where f = average rainfall in inches; m = catchment area in sq. miles; h = average head in feet.

1 kW. at the customers' premises requires 1·364 gallons of water (5·163 litres) at average head of 590 feet (179·8 metres).

In our country the excess of water-power that may be derived during monsoon period cannot be calculated, the water is seldom stored and the whole available power is averaged throughout the year and the maximum power is also the actual.

"Ordinary minimum flow is based on the averages of the minimum flow for the two lowest consecutive seven-day periods in each year over the period for which records are available."

"The estimated flow for maximum development is based upon" the continuous power indicated by the flow of the stream for six months in the year. The actual method to determine this flow is to arrange the months of each year according to the day of the lowest flow in each; the lowest of the six high months is taken as the basic month; the average flow of the lowest seven consecutive days in this month determines the maximum for that year. The average of such maximum figures for all the years in the period for which data are available is the estimated maximum used in the calculation."

The measurement of a discharge.—Wherever a stream offers possibilities of water-power, it is necessary to ascertain the minimum flow of water in it; this, however, may involve waiting for years, as it is only after an exceptionally dry season or succession of dry seasons that the true minimum is reached; and as this was the case in many parts of India in the cold weather of 1918, it may not recur for a long time. A cycle of about 30 years is required to determine the definite minimum. The minimum discharge of any year is, however, of value, as some indication of the true minimum may then be obtained by examination of rainfall records in the neighbourhood, especially if the average run-off is known. (To the skilled observer the method of taking a discharge is familiar). Many persons accustomed to this work may, however, be able to render invaluable help by estimating the flow of streams in out-of-the-way places. The amount of water in cubic feet per

* W. T. Taylor, "The Electrical Review," October 12, 1923.

second (cusecs) flowing past a given point in any open channel is found by multiplying the cross-sectional area of the water in square feet by its mean velocity in feet per second.

Where the correct methods and coefficients to be employed are unknown, the following very approximate method may be adopted :—

Select a straight reach of the stream where the flow is uniform and where there are no noticeable pools, and mark off with pegs any convenient length of fairly uniform width. Then measure as accurately as possible the width of the stream, and find the average depth by soundings at intervals across the stream. Then throw into the centre of the stream a small stick and note the time it takes to pass along the marked length; repeat this 4 or 5 times to eliminate errors. Then the run or length in feet divided by the average time in seconds gives the velocity on the surface. Then the cross-sectional area in square feet multiplied by the surface velocity in feet per second and *divided by two* will give the approximate discharge in cubic feet per second.

For instance, suppose the length marked off is 100 feet and the floats take an average time of 50 seconds in passing along this length in a stream having a width of 40 feet and an average depth of $1\frac{1}{2}$ feet. This gives a cross-sectional area of 60 square feet. The surface velocity will then be 2 feet a second and the discharge $\frac{2 \text{ feet a second} \times 60 \text{ square feet}}{2} = 60$ cusecs. The reason for dividing by 2 is that the mean velocity throughout the channel is much less than the surface velocity, and the rougher and shallower the bed may be the greater will be the disproportion ; in the worst cases it is about half.

"It sometimes happens that soundings cannot be taken in unfordable hill streams, and another very rough method which has proved of occasional use may be mentioned. Gaugings were requisitioned of a river, partly snow-fed, at its junction with a smaller hill stream. The discharge of the latter was satisfactorily determined, but that of the former could not be measured. The temperature of the larger river was found to be 60° ; that of the smaller one 64° ; and of the combined river some little way down stream 61°. Then the volume of the large stream is thrice that of the smaller, for—

60 × 3 volumes = 180,
64 × 1 volume = 64.
4 volumes give 244 or 1 volume gives 61° as given in data."

Hence, we see that the volume of the large stream is thrice the smaller.

A horse-power year or a kW.-year is 8760 times a h. p.-hour or kW.-hour.

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For instance, suppose the length marked off is 100 feet and the floats take an average time of 50 seconds in passing along this length in a stream having a width of 40 feet and an average depth of $1\frac{1}{2}$ feet. This gives a cross-sectional area of 60 square feet. The surface velocity will then be 2 feet a second and the discharge $\frac{2 \text{ feet a second} \times 60 \text{ square feet}}{2} = 60$ cusecs. The reason for dividing by 2 is that the mean velocity throughout the channel is much less than the surface velocity, and the rougher and shallower the bed may be the greater will be the disproportion ; in the worst cases it is about half.

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4 volumes give 244 or 1 volume gives 61° as given in data."

Hence, we see that the volume of the large stream is thrice the smaller.

Basis of Measurement of Stream Flow

The ordinary minimum stream flow is frequently taken as the safest basis upon which to draw up water-power estimates. In general, the more important problem is to determine the proper or economical amount above the ordinary stream flow for which the development should be built. If absolute continuity of service is guaranteed and there is no storage and no interconnection with other power plants, it is best to take ordinary minimum flow and to use it in connection with maximum load anticipated.

Methods of Measuring Stream Flow

I. *Direct Discharge Methods*:—Hereby the total discharge is determined directly, independent of any knowledge of the velocity or sectional area and embraces such means as (*a*) weirs, (*b*) chemical gauging, (*c*) volumetric or bulk method.

II. *Mean Velocity Area Method*:—(*a*) Methods whereby the mean velocity of the entire stream can be determined directly by one operation such as :—

(1) Slope determination.
(2) Colour or brine methods.
(3) Travelling screen.
(4) Orifice or nozzle methods.
(5) Venture meter, etc.

(*b*) Methods whereby the total cross-sectional area is divided into sections whose mean velocities, areas and discharges are summed up to give the total discharge of the stream. These methods differ only in the manner of determination of mean velocity of the section and they may comprise the use of one or more of the following means :—

(1) Current meter.
(2) Rod float.
(3) Surface or subsurface floats of which rod float is one, and
(4) Pitot tubes.

SELECTION OF GAUGING METHODS

Accuracy good for streams if measurement is within 4 to 9 %.
Accuracy good for conduits if measurement is within 1 to 3 %.

	Condition of stream.	*Gauging method recommended.*
1.	Small streams ...	Current meter or sharp-crested weir.
2.	Open channels of uniform section	Current meter or rod float.
3.	Narrow, deep and uniform section	Current meter rod float, colour or brine method.
4.	Wide and shallow stream ...	Current meter.
5.	Deep and narrow stream ...	Current meter or chemical process.
6.	Swift or turbulent mountain stream	Chemical process and sometimes current meter.
7.	For closed conduits ...	Salt solution or chemical method venture meter or Pitot tube.
8.	For open conduits ...	Weir current meter, chemical process.

Measurement of Stream Flow

Two methods are ordinarily used for determining the quantity of water flowing in open channels—(1) the velocity area method, (2) the weir method.

. The method to be adopted in gauging stream depends on the size of the stream, its condition and on the accuracy that is desirable in each particular case. The accuracy depends upon the physical characteristic of the stream at the point of measurement. A weir, if possible, should be located, so that smooth flow, free from eddies, surface disturbances, or the presence of considerable quantities of air in suspension, exists in the channel of approach. The uniformity of velocity should be verified by current meter. For a stream of medium size the rectangular weir is most desirable. For a small flow the triangular notch has advantages.

The simplest, most accurate, reliable and the most satisfactory method for a small stream is by means of a weir. A vertical obstruction is placed in the stream bed containing a notch of either trapezoidal, rectangular or V form over which the water falls. A dam is made extending across and at right angles to the stream. A rectangular notch is cut in the top plank with both side edges and bottom sharply bevelled towards the intake. The bottom of the notch, called the crest of the weir, should be perfectly level and the sides vertical. The length or width should be between four and eight times the depth of water flowing over the crest of the weir. The pond back of the weir should be at least 50 per cent. wider than the notch and of such width and depth that the velocity of flow or approach be not over one foot per second.

The standard sharp-crested weir used in most countries is of a metal crest (of steel plate) with a sharp right-angled corner on the upper stream edge, a crest with 1·8th of an inch bevelled to an angle of 45° on the downstream face. Knowing the dimension of the weir, the observation necessary for the determination of the discharge is the head or depth of water flowing over the crest. It is important that the channel of approach be straight and uniform and that the crest be carefully levelled.

On the upstream side, a stake is driven down in the bottom of the pond, near the bank, so that its top is level with the bottom edge of the notch, this level being easily found when the water is beginning to spill over the crest. The stake should be placed several feet from the board at least as far from it as the length of the notch.

The depth of the water over the top of the submerged stake is measured by means of a rule. Allowance is made for the capillary attraction of water against the side of the weir. When the depth is ascertained, the amount of water flowing over the weir may be readily found. The author used this method to measure the discharge of the stream at Phurping, Nepal—the fall is about 700 ft. falling down the bank of the Bagmati River. Then apply the formula :—

Rectangular weirs—

$$Q = 3{\cdot}33\ (L - 0{\cdot}2\ h)\ h^{\frac{3}{2}} \text{ with double-end contraction.}$$

$$\text{or}\quad Q = 3{\cdot}33\ (L - 0{\cdot}2\ H)\ \left[(H - h)^{\frac{3}{2}} \times h^{\frac{3}{2}} \right]$$

$$Q = {\cdot}0801\ (L - H/60)\ H^{\frac{3}{2}} \text{ without end contraction.}$$

Trapezoidal weirs (Cippoletti) $Q = 3{\cdot}367\ Lh^{\frac{3}{2}}$

$$= {\cdot}081\ LH^{\frac{3}{2}}.$$

V notch (right-angle) weirs $Q = 2{\cdot}54\,h^{\frac{3}{2}}$.

$Q = 2{\cdot}48\ H\ 2{\cdot}47$ (vertical angle 90°)

$$Q = 0{\cdot}0611\ H^{\frac{3}{2}}.$$

where Q = cubic feet of water per second.
h = head or height over crest or sill in feet.
L = length of crest or sill in feet.
H = head in inches.

Weirs carefully used under favourable circumstances are accurate within 1 per cent.

Where measurement by weir is not possible, the quantity of water can be determined by ascertaining the average velocity of the water and the cross-section of the stream ; the quantity is the product of these two. The mean velocity is the function of the cross-section, surface slope, wetted perimeter, roughness of the bed, while the cross-sectional area depends on the permanence of the bed and the fluctuation of the surface which governs the depth.

To measure large streams it is too costly to construct a weir only for the temporary purpose of measurement. If a permanent weir is not already available, the discharge is measured by taking the mean velocity of the stream and multiplying it by the area of the cross-section. The velocity-area method is now almost universally used in river gauging because of its comparative cheapness and its essential reliability and applicability under

a wide range of conditions, and because of the excessive cost of constructing and rating weirs or dams for river gauging only and the inaccuracies involved in using any except standard sharp-crested weirs with free overfall.

The object of the Cippoletti weir is to eliminate the effect of end contractions. The weir has a top width of 1·3 L with bottom width of L and a depth of 0·6 L with side slopes of 1 in 4.

The mean velocity is obtained by any of the following methods :—

(1) **By current meter** :—Current meter gives very accurate readings of mean velocity in the period of observation. In this method measurement along a single cross-section is sufficient. Five to ten minutes is required for each observation.

By current meter, take the velocities at equal vertical intervals at about 12 inches or more and obtain their arithmetical mean.

By current meter, take the velocity at 0·6 of the water depth. By the use of this method with bottom and sides of smooth and uniform section for a sufficient distance both up and down stream (for normal stream flow of moderate depth) the use of a coefficient is unnecessary in the determination of the mean velocity. This method is usually reliable over a wide range of conditions, and in North America it is in common use. (a).

By current meter, take the velocity at 0·2 and that at 0·8 of the water depth and obtain one-half the sum. This method is used for general stream gauging. When closer accuracy is required, two current meters should be used, one for the method (a) and one for this method. The degree of accuracy will depend on the ratio of width to depth and the velocity.

By current meter, take the velocity at 0·2, 0·6, and 0·8 *depth*. The mean velocity can be obtained by *dividing* by four the sum of the measured velocities at 0·2 and 0·8 depth *plus* the velocity at 0·6 depth. The mean velocity can also be computed from the *approximate* formula :—

$$V = \frac{\dfrac{k\,(V' + V'' + V'''')}{2 + V'''}}{3}$$

V' = surface velocity.
V'' = velocity at 0·2 of the depth.
V''' = velocity at 0·6 of the depth.
V'''' = velocity at 0·8 of the depth.
k = coefficient varying from 0·85 to 0·95, according to the depth, width, and velocity.

By current meter, take the velocity near the water surface and use from 0·80 to 0·95 of the result. The value of this coefficient will depend on the depth of the water: its velocity, and the nature of the bed of the stream. The point of observation should be from 6 to 12 in. below the surface, its proper location depending on the depth of the stream. This method is generally used in swift flowing streams and in streams during times of freshet.

By current meter, take velocity at 0·5 depth. The *approximate* formula for mean velocity is : $V = V' \times 0{\cdot}96$, wherein V' = velocity at mid-depth.

By current meter, take velocity in the vertical line by uniformly lowering and raising several times the current meter throughout the range of water depth. This method may also be used to determine the coefficients given in some of the above-mentioned methods.

By current meter, take six intervals of depth, giving seven velocities, including the surface velocity V_1 and the bottom velocity V_7, the mean velocity in the vertical is then computed from formula—$V = \frac{1}{18} [V_1 + V_7 + 4 (V_2 + V_4 + V_6) + 2 (V_3 + V_5)]$

There are various types of current meters. The most generally used is the Price meter. It cannot be used in water having floating grass or weed, and is not suitable for very low velocities. It requires rating at frequent intervals. The velocity of the flowing water is measured at several points along a single cross-section. Each measured velocity is multiplied by the appropriated partial area to obtain the partial discharge. The error is not to exceed 5 per cent. and in good work it will be less than 2 per cent. due to :—(1) rating of the meter, (2) observations of soundings, (3) in placing the meter in position, (4) in observations of the revolution of meter, (5) in observations of time, (6) in the use of insufficient observations of velocity.

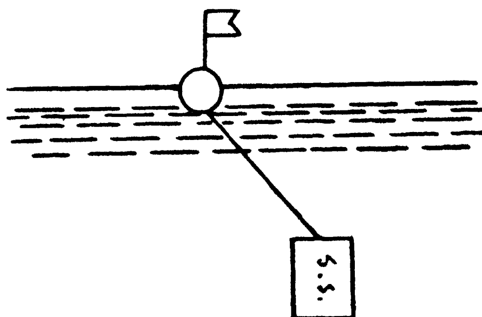
(2) **By floats** :—These are surface-floats, subsurface-floats, or a combination of the two, and rod floats.

The floats are of three types :—

(*i*) **Surface-floats.**—A number of small floating bodies which move with the register; the velocity of the surface flaments are calculated at a series of points across the stream and the time occupied to flow a distance of about 200 feet is measured. The mean velocity is obtained by multiplying the surface velocity by a factor which varies from ·85 to ·95 according to the condition of the channel. An orange or an empty bottle sealed up with a weight placed in the bottom or a small piece of wood serves very well. Nearly all surface floats are useless, as they

give only surface velocity and with almost any wind blowing, the velocity cannot be measured with accuracy.

(*ii*) **Sub-surface floats.**—These have large area of surface and carry a small float at the surface for observation. The length of the float is adjusted to keep the true float at any desired depth. The velocity is that of the current at the required depth. A number of floats may be used at different depths along the cross-section of a stream and their mean velocity will indicate the mean velocity of the stream. As a rule, sub-surface float is unreliable as it is easily caught by eddies and cross-currents which move it about.



The error is due to (1) the difficulty of determining the depth of the lower float, (2) unequal velocity of the upper and lower filaments of water at different depths. The percentage of error varies from 5 to 25 %.

(*iii*) **Rod floats** are made of light wooden rods or tin tubes about 1 inch in diameter and in adjustable lengths. The lower end of the bottom section is weighed and the length adjusted until the rod floats vertically with its lower end clearing the bottom by a few inches. The velocity of the rod is approximately the same as the mean over its depth and gives the mean velocity over the vertical in which it floats.

In channels of moderate and uniform depth this method gives very good results. This is the method demonstrated to the students of the Engineering College to take the discharge of the Ganges Canal at Roorkee.

The difficulty lies in its tendency to drag over shoals and weeds, and to obviate this, its lower end may be arranged to float at some height above the bed of the stream.

Due to the pulsation of velocity existing normal in a steady flow, a light float may give very unsatisfactory results.

There are errors due to the effect of the wind, cross-current and eddies.

Average of a large number of observations must be taken. This method is not good for channels of irregular section or in rising and falling stream.

The mean velocity by means of the rod float is computed by the approximate formula :—

$$V = V^{\circ} \left[1{\cdot}0 - 0{\cdot}116 \left(\sqrt{D} - 0{\cdot}1 \right) \right]$$

where V = mean velocity in vertical,

V° = observed rod float velocity,

D = proportion of depth not reached by the rod.

Francis Lewitt's hydraulic experiments give :—

$$V = V^{\circ}\ 1{\cdot}012 - 0116 \left(\sqrt{D} \div D' \right)$$

where D' = depth of stream.

D = clearance between the bottom of the rod and the bed of the stream.

This clearance should be less than $0{\cdot}25\ D'$.

The mean or average velocity can also be approximately determined from the formula :—

$$V = A\ (L/t)\ K,$$

wherein L = length in feet ; t = average time in seconds ; K = coefficient = 0·75 to 0·85 ; A = calculated cross-sectional area.

The float method is more accurate than the slope method, but is less accurate than the current meter method. It may be used without special equipment and in rivers carrying so much draft that a current meter method cannot be used. The error may be 15 per cent. or more and may occur in three ways :—(*a*) in failure to obtain a true average area of cross-section of the stretch of river used ; (*b*) in failure to obtain properly distributed observations of velocity, and (*c*) in choosing a wrong coefficient to reduce observed velocities to mean velocities.

(3) **By colour or chemical method** :—This takes a little time ; no delicate apparatus is necessary and is much better than the float method, but the injection must be sufficient to colour an appreciable portion of the cross-section and the section must be regular.

Colour velocity.—Inject colouring matter into the stream in a single burst. Note the time it takes to traverse a measured distance. Note at the down stream measuring point the times at

which the first and last traces of colour pass, also the time at which the point of maximum colour passes. Take the mean of the 1st two readings which should agree with the third. Permanganate of potash may be used in clear water, and good vegetable stains, red or green aniline dye gives good result. This method is well adapted to measurement of the flow in a long penstock.

(4) **The colour density method** :—The colouration or colour density may also be employed for approximate tests. This method depends on the use of coloured dosing solution, observation of the colour density replacing the trituration.

Chemical method :—A strong solution of some chemical, such as salt, caustic soda or sulphuric acid, gives good results if the stream is wide and preferably near the mid-path. It is added, at a number of points in a cross-section at a uniform known rate, into a stream. Then at a certain point down the stream or in tail race, samples are collected and analyzed from some point below where admixture is complete, the degree of dilution is determined and the volume is calculated. If p pounds of salt be added each second to a stream whose discharge is Q cusecs and after a thorough mixture it is found that x pounds of water contain 1 lb. of salt, the total discharge Q will roughly represent x p/62·4 lb. The salt is introduced as a solution of about 16 lbs. per c. ft. of water and the rate should be such that the water of the stream after admixture contains not less than 1·0 part of salt in 75,000 parts. Sufficient solution should be used to give a uniform supply from 20 to 30 minutes. The accuracy depends upon the samples being truly representative of the water at the collecting station.

This method is very suitable to measure the rapids and irregular streams in which admixture is most thorough and which are most difficult to gauge by other means.

A surface float gives the approximate time necessary for the 1st of the mixture to reach the sampling section. Take the sample after a few minutes, and immediately before and after the tests, a sample of stream water should be tested for acidity or salinity and the result of the mixture tests should be corrected for the presence of any natural salinity or acidity, if present.

The Resistance of Salt Solution :—In this the amount of chemical salt in solution is determined by measurement of the electrical resistance of the solution instead of by trituration. Care is necessary to guard against change in resistance due to small temperature variations.

(5) In the case of an artificial channel of uniform section the mean velocity may be determined by *measurement of the slope*

of the stream. In this method the determination of friction factor, from the cross-section and measured slope, etc., there is cause of probable inaccuracy due, not only to the roughness of stream bed and banks, but to an inefficient cross-section and slope as well as the engineers' judgment of channel condition. The value of n may also differ between flood and low flow of the stream. For the stream flow computation use Manning's formula :—

$$V = cR^{\frac{2}{3}} S^{\frac{1}{2}} = cR^{.67} S^{.5} = c\sqrt[3]{R^2}\sqrt{S}\text{; and } Q = cAR^{\frac{2}{3}} S^{\frac{1}{2}},$$

wherein $c = 1{\cdot}4867/n$, where R is the hydraulic radius.

" For nearly all except very low discharges the value of n in this formula is a fairly good constant. Kutter's formula gives too large values for small slopes. In using the Kutter formula it is customary to assume a value for n for a stretch of stream channel and to use the same value regardless of change in slope or in hydraulic radius. Also, in fixing the value of n the character of the channel alone is considered in the Kutter formula, the slope usually being disregarded so far as its effect on n is concerned. The values of n to be used in the Manning's formula for calculating values of c in $V = (1{\cdot}4867/n)\ R^{\frac{2}{3}} S^{\frac{1}{2}}$, are :—

Coefficient of roughness n to be used for different conditions of Channel.

Serial Number.	Value of c.	Value of n.	Conditions of Channel.
1	29·7	0·0500	For large streams in flood ; mountain streams.
2	49·5	0·0300	For moderate size streams with loose stone bottoms, relatively shallow, with very irregular course and sharp bends.
3	59·6	0·0250	For streams (not large) with smooth sandy bottoms, relatively narrow and deep, with irregular channel.
4	66·0	0·0225	For earth sections (conduits) in fairly good condition.
5	74·3	0·0200	For earth sections (conduits) lined or cobble stone section.
6	87·5	0·0170	Canals with gravel bottoms and sides well rammed.
7	99·0	0·0150	Rock-cut and tunnel section fairly smooth (good).
8	114·3	0·0130	Tunnels and other sections, concrete or timber lined (good).

For the different character of channels, values of n will vary somewhat from the values given above. For instance, (1) and

* P. 87, Practical Water Power Engineering by W. T. Taylor.

(2) in the above table may be greater, but rarely less. Also, the value of 0·025 may be high for the *ordinary* earth section conduit. For rock-cut unlined sections, the value of η will usually lie between 0·025 and 0·030, but this will depend on the amount of trimming. Also, for tunnels (concrete-lined) η may be 0·0125 or even 0·0145.

(6) By Pitot Tube :—

Principle—It is a glass tube having a curved bend in it. The two branches are at right angles to each other. One end of the tube is immersed in water, pointing upstream in the direction of the current, while the other end projects vertically upward from the water surface. The height h in feet to which the water rises in the vertical branch above the surface of the water through the orifice which should be very small, say $\frac{1}{6}''$ to $\frac{1}{8}''$ diameter, gives the indication of the velocity v in feet of the flow.

$$h = v^2/2g.$$

To measure streams having rough beds and in which the velocity is, however, constant, the level of the water in the two tubes will oscillate and reading is difficult. To overcome this there are short sections of the tubes where the diameter is greatly increased, a mass of water is stored in them which has sufficient inertia to prevent the fluctuations in velocity from producing oscillations of the water columns in the tubes whose heights are the averages of the changing velocities at the mouth of the impact tube.

The pitot tube is not useful for practical measurements when the velocity is below 2 feet per second. Mr. White's experiments have shown that the tube must be smooth inside and out, the orifice must not have burs or swells and it should be without any changing in the walls of the tube. (" White's Journal of Association of Engineering Society, August, 1901 ".)

CHAPTER VI

HYDRAULICS

Flow through Pipes, Channels, Weirs, etc.

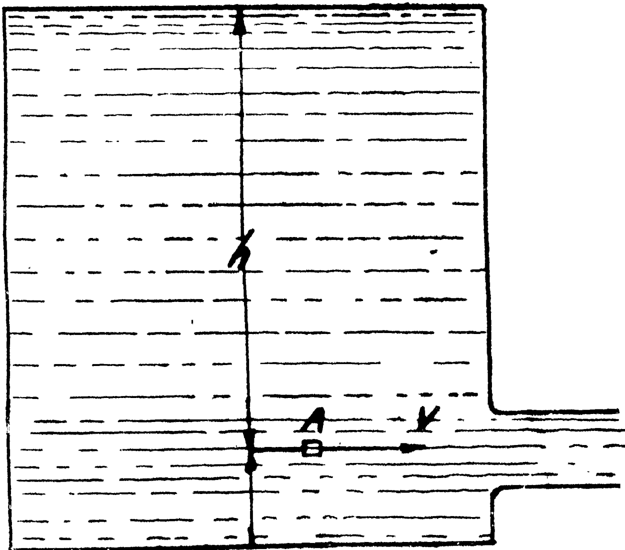
Energy of water :—The universal law of the conservation of energy is applicable to the particles of water in motion. The water in motion possesses energies due to its motion and position, and the same can be classified as follows :—

(*i*) Kinetic energy,
(*ii*) pressure energy,
and
(*iii*) potential energy.

As the head of water is the measure of pressure at any point in a liquid, the three forms of energies are also measured in terms of the head of water.

Consider a point A at a depth of h feet from the surface of water and H feet above the datum-line.

Fig. 1.



Let v ft./sec. be the velocity of water at A, and P lbs./sq. ft. be the water pressure at A. The kinetic energy per lb. of water at A is equal to $v^2/2g$ and it is equivalent to energy possessed by the water in falling freely through a distance $v^2/2g$ ft. $v^2/2g$ is called the *kinetic head of water*. The *pressure energy* of the water at A is P/62·4 and it is the height of the column of water which would produce the pressure P at the base.

P/62·4 is called the *pressure head of water*. The potential energy of the water at A in terms of the head of water is H and it is due to its position above the datum line, as the water can

descend vertically up to the datum line. H is called the *static head of water.*

Hence energies of water at A can be computed as follows :—

Kinetic energy per lb. of water = $v^2/2g$
Pressure ,, ,, ,, = P/62·4
Potential ,, ,, ,, = H.

Bernoulli's Theorem :—For any mass of water in which there is continuous connection between all the particles, the total energy of each particle is the same. This is known as Bernoulli's Theorem and this is the basic theorem of hydraulics. According to this if water flows from a point (a) to another point (b) at lower level :—

$$\frac{P_a}{W} + H_a + \frac{v^2_a}{2g} = \frac{P_b}{W} + H_b + \frac{v^2_b}{2g}$$
$$= \text{Constant.}$$

If there be friction on the path of flow, let h_f be the head lost in friction per lb. of water. The equation of energy reduces to : —

$$\frac{P_a}{W} + H_a + \frac{v^2_a}{2g} = \frac{P_b}{W} + H_b + \frac{v^2_b}{2g} + h_f.$$

The surface of water is taken as the datum pressure line and hence the pressure heads due to atmospheric pressure will cancel one another from the two sides of the equation under Bernoulli's Theorem.

Hydraulic Losses :—While water is flowing through a pipe, the following losses of head are to be considered :—

(*a*) Loss of head due to friction.
(*b*) Loss of head due to bends and elbows.
(*c*) Loss of head due to variation of the pipe section.
(*d*) Loss of head due to obstruction.

Friction losses in pipe-lines :—According to the modified form of Chezy formula, head lost due to friction in pipe is given by—

$h_f = f\, l\, v^2/2g\, m$, f t.

where l = length of the pipe line in feet.
v = velocity of the flow of water.
f = constant, found from experiment.
m = hydraulic mean depth or radius.
$= \dfrac{\text{cross-sectional area of the pipe}}{\text{wetted perimeter}}$.
d = diameter of the pipe in feet.

If the pipe runs full of water,

$$m = \frac{\pi d^2}{4} \times \frac{1}{\pi d} = \frac{d}{4}$$

∴ head lost due to friction in pipes running full of water,

$$h_f = \frac{4 f l v^2}{2 g d}.$$

D'Arcy gives the following equation for determining the value of f :—

For new pipes $f = \cdot 005 \left(1 + \frac{1}{12 d}\right)$

For old pipes $f = \cdot 01 \left(1 + \frac{1}{12 d}\right)$

The most recent investigations show an exponential of the type

$$h = \frac{f l v^n}{d^x} \text{ ft.}$$

and this formula generally agrees with the experimental results.

The approximate **loss of head in a pipe** in feet is as follows :—

Let d = diameter of riveted pipe in inches.
V = velocity of flow in feet per second.
L = length of pipe in feet.
h = head lost in feet.
$h = L (v^2 + 1{\cdot}25v - 0{\cdot}5)/250\, d.$

Loss of Power in Conduit

Q = discharge in cusecs.
L = length of conduit in feet.
S = grade or slope in feet per foot of conduit.
e = Efficiency of water (from head works intake to turbine shaft or at switch board) multiplied by ·0846 k.W.

Power loss $p = QLSe$, $SL = h$ and $S = h/L$.

$\therefore P = Qhe$.

For various materials Mr. A. A. Barnes gives the following values of f, n and × from various experiments.

Material.	Value of h_f in ft.	Mean velocity fs.	Quantity of discharge in cubic ft. /sec.
New uncoated cast-iron pipes.	$\frac{0{\cdot}000343\ l\ v^{1{\cdot}953}}{d^{1{\cdot}172}}$	$136{\cdot}6\ m^{{\cdot}6} \cdot i^{0{\cdot}51}$	$46{\cdot}7\ d^{2{\cdot}69} \times i^{{\cdot}512}$
New asphalted cast-iron pipes.	$\frac{0{\cdot}000436\ l\ v^{1{\cdot}891}}{d^{1{\cdot}454}}$	$174{\cdot}1\ m^{{\cdot}769} \cdot i^{{\cdot}529}$	$47{\cdot}1\ d^{2{\cdot}769} \times i^{{\cdot}529}$
New asphalted single-riveted wrought-iron and steel pipes.	$\frac{0{\cdot}000386\ l\ v^{1{\cdot}898}}{d^{1{\cdot}372}}$	$171{\cdot}4\ m^{{\cdot}723} \cdot i^{{\cdot}527}$	$49{\cdot}4\ d^{2{\cdot}723} \times i^{{\cdot}527}$
Do. double-riveted with taper or cylinder joints.	$\frac{0{\cdot}000279\ l\ v^{1{\cdot}923}}{d^{{\cdot}846}}$	$129{\cdot}9\ m^{{\cdot}44} \cdot i^{{\cdot}52}$	$55{\cdot}4\ d^{2{\cdot}44} \times i^{{\cdot}52}$
New smooth wood-stave pipes.	$\frac{0{\cdot}00047\ l\ v^{1{\cdot}707}}{d^{1{\cdot}126}}$	$223{\cdot}3\ m^{{\cdot}66} \cdot i^{{\cdot}586}$	$71{\cdot}3\ d^{2{\cdot}66} \times i^{{\cdot}586}$
New unplaned wood-stave pipes.	$\frac{0{\cdot}000541\ l\ v^{1{\cdot}757}}{d^{1{\cdot}171}}$	$182{\cdot}5\ m^{{\cdot}666} \cdot i^{{\cdot}569}$	$58{\cdot}5\ d^{2{\cdot}666} \times i^{{\cdot}569}$
Clean neat cement pipes.	$\frac{0{\cdot}00024\ l\ v^{2{\cdot}066}}{d^{1{\cdot}312}}$	$136{\cdot}3\ m^{{\cdot}635} \cdot i^{{\cdot}484}$	$42{\cdot}0\ d^{2{\cdot}635} \times i^{{\cdot}484}$

Loss of head due to bends and elbows :—The loss due to a right-angled bend depends upon its radius of curvature. In practice the best radius used varies from 2·5 to 5 times the diameter of the pipe. For such bends loss of head is nearly $0·3v^2/2g$ ft.

If the bend is carried round an angle θ which is less than 90°, the loss is approximately proportional to θ^2.

Kellog Company's bulletin gives $h_b = f_1 \times \frac{b}{180} \times \frac{V^2}{2g}$.

hf = loss of head in bends
b = angle of the bend in degrees
where f_1 = curve factor determined from the relation R/d, R being the radius of the bend ;
d = the pipe dimeter.

For R/d =	5	4	3	2	1	0·75	0·5
f_1 =	0·13	0·13	0·14	0·14	0·3	0·6	2·0

Loss due to variation in pipe-section :—The variation of the pipe-section may be affected in two ways, *viz.* :—sudden enlargement, and sudden contraction.

The loss of head of water due to sudden enlargement of the pipe-section is equal to $(v_1-v_2)^2/2g$ ft. where v_1 and v_2 are the velocities of water in feet per second in the smaller and the bigger pipe-sections, respectively.

The loss of head of water due to sudden contraction of the pipe-section is equal to $0·5v^2/2g$ ft., where v is the velocity of water in feet per second in the contracted pipe-section.

The loss of head of water while entering a pipe from a large container is actually a loss due to sudden contraction and if v be the velocity of water in the pipe, the loss of head = $0·5v^2/2g$.

Loss of head due to obstruction :—The loss of head due to obstruction in a pipe is given by :—

$$\left[\frac{A}{0·66\,(A-a)}-1\right]^2 \frac{v^2}{2g} \text{ ft.}$$

where A = cross-sectional area of the pipe.
a = area of the obstruction in the pipe.
v = velocity of water in the free section of the pipe.

Summary of Losses of Head

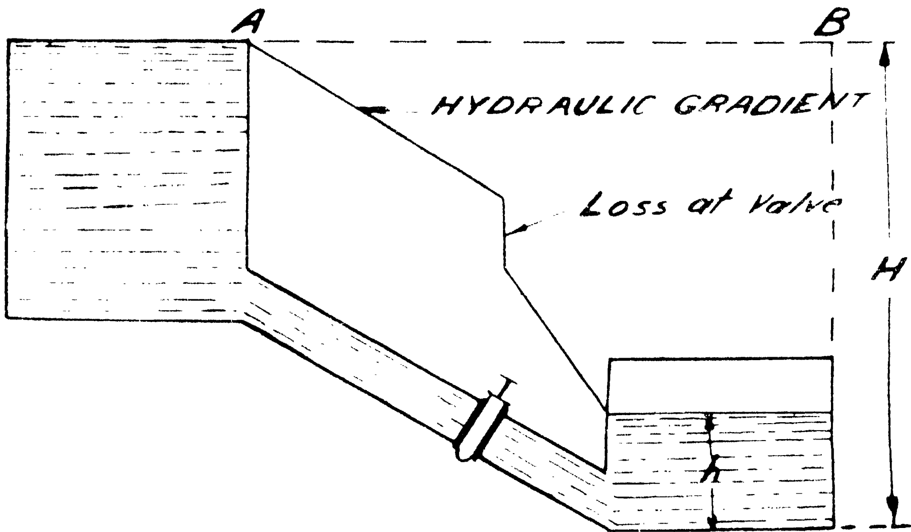
(1) Loss of head due to friction in pipe $=4flv^2/2gd$.
(2) Loss of head due to bends or elbows = $0·3v^2/2g$.

(3) Loss of head due to sudden enlargement $= (v_1 - v_2)^2/2g$.
(4) Loss of head due to sudden contraction $= 0{\cdot}5\,v^2/2g$.

(5) Loss of head due to obstruction in pipe $= \left[\frac{A}{0{\cdot}66(A-a)} - 1\right]^2 \frac{v^2}{2g}$.

Hydraulic Gradient :—Let AB be the line to represent the free surface of water. If ordinates be drawn downwards from A B to represent to scale the total loss of pressure head from the pipe entrance to the particular point considered, the ends of such ordinates being joined give a curve known as the *Hydraulic Gradient.* For the definite pipe, the loss of head at any point is proportional to the length of the pipe measured from the pipe entrance to the very point and hence the hydraulic gradient will be a straight line up to the point.

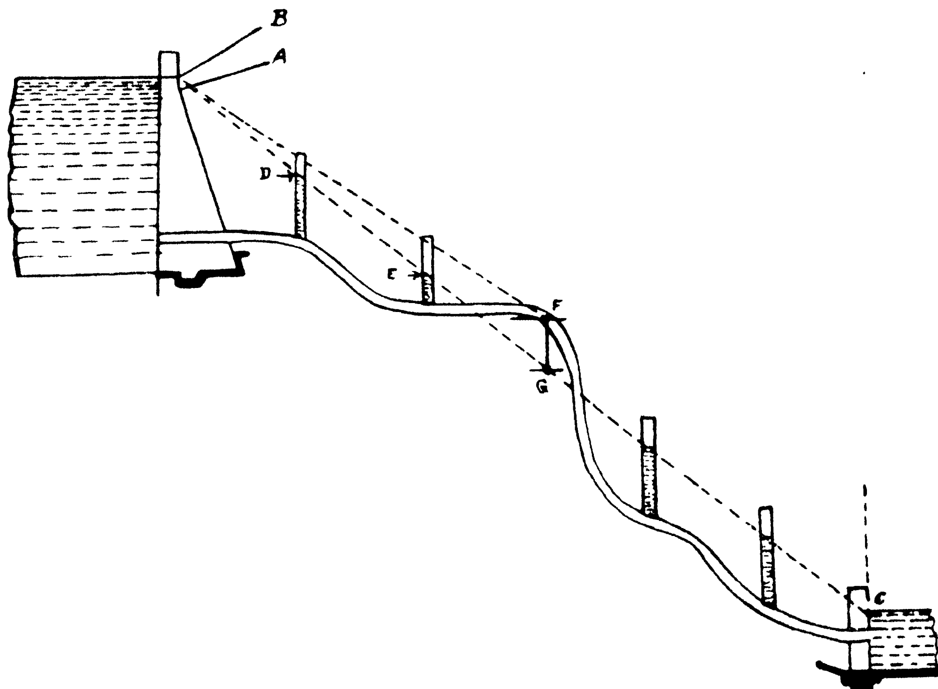
Fig. 2.



From the conditions of flow it is clear that there is a continual absorption of head to maintain the flow and this is proportional to the length of the pipe.

In Fig. 3 AB is equal to the velocity head *plus* the entrance head to a point at the end of the pipe $= (1+0{\cdot}5)\,V^2/2g$. For a pipe discharging freely in the air this would be the centre of the outlet ; but for a pipe with submerged discharge it would be the lower water-level instead of the point of discharge.

Fig. 3.



The slope or drop in elevation along the pipe corresponds to the friction loss. The vertical distance between D and E would be equal to the head lost on account of friction between these two points.

If the pipe is so laid that it rises above the hydraulic gradient AC, as at F, the pressure in the pipe at this point will be less than that of the atmosphere by a head corresponding to GF; thus it will be negative. If no air could enter the pipe, it would act as a siphon, and the flow would continue as usual provided the distance FG did not exceed about 25 feet, the theoretical value of the vacuum being 34 feet.

But air is always present in water and it will collect at the summit near F with the result that the pressure will approach atmospheric pressure. In this case the gradient would shift to AF and the discharge would only be that due to the vertical head between B and F instead of between B and C. The remainder of the pipe from F to C would merely act as a channel to deliver the flow.

Thus it is evident that the pipe line should be laid below the hydraulic gradient. Much trouble may be avoided if, at the outset, a profile of the proposed route is prepared and the hydraulic gradient carefully calculated and drawn in.

In order to prevent difficulties arising from liberation and accumulation of air at such points and from admission of air at leaky joints, the greatest height above the gradient line should never exceed 20 ft. Owing to weakness, large pipes under water pressure should not be laid above the gradient line.

To determine discharge through the pipe we can apply Bernoulli's Theorem, when—

$$H = h + \frac{v^2}{2g} + \frac{f\,l\,v^n}{d^x}$$

from which the value of v can be calculated, then

$$\theta = C_d\, A \times v$$

where θ = quantity of discharge,
C_d = coefficient of discharge,
A = area of the cross-section of the pipe,
v = velocity of flow through the pipe.

Power Transmission Through Pipes

Let A = total head supplied at entrance to pipe,
h_f = head lost due to pipe-friction,
l = length of the pipe,
d = diameter of the pipe,
v = velocity of the flow of water in the pipe.

$$\therefore \quad h_f = 4\,f l\, v^2 / 2\,g\,d.$$

Total head available at the outlet of the pipe.

$$= H - 4fl\, v^2/2\,g\,d.$$

$$\therefore \text{ Available horse-power} = \frac{\pi d^2 \times v \times 62{\cdot}5}{4 \times 550}(H - h_f)$$

$$\text{or H. P.} = \frac{6{\cdot}25\,\pi\, d^2}{4 \times 550}\left(Hv - \frac{4\,fl\,v^3}{2\,g\,d}\right)$$

$$\frac{d\,(H.P.)}{d\,v} = H - \frac{12\;fl\;v^2}{2\,g\,d}$$ and for the H.P. to be maximum, this $\frac{(H.P.)}{d\,v}$ should be equal to zero.

$$\text{or } H - 3\,h_f = o, \text{ or } h_f = \frac{H}{3}.$$

Hence, maximum horse-power is transmitted by a pipe when frictional loss of head is one-third of the total head supplied.

The velocity head at the outlet end of the pipe = $v^2/2g = \frac{2}{3}$ H.

Hence, for maximum efficiency of transmission—

$$h_f = \tfrac{1}{3}\,H$$
$$= \tfrac{1}{2} \times v^2/2\,g$$

= half the velocity head transmitted.

Example :—A hydraulic machine is supplied with water through a horizontal pipe 3000 ft. long. The brake horse-power of the hydraulic machine is 50 and its mechanical efficiency is 80 per cent. Gauges fitted to the supply pipe show that the pressure at the power station end is 750 lbs. per sq. in. ; and at the machine, 680 lbs. per sq. in. If the coefficient of resistance for the pipe is ·008, determine (1) the diameter of the supply pipe, (2) the velocity of flow (London University, 1917).

Let A, D, V be the area, diameter and the velocity of the supply pipe, respectively.

H. P. supplied by the machine—

$$= \frac{50 \times 100}{80} = 62{\cdot}5,$$

and since the head is $\frac{680 \times 144}{62{\cdot}4}$ ft.,

the H. P. should be equal to $\frac{62{\cdot}4\ A\ V}{550} \times \frac{680 \times 144}{62{\cdot}4}$

$$A\ V = {\cdot}35 = \frac{\pi}{4} D^2\ V$$

$$\text{or } D^2\ V = {\cdot}445 \ldots\ldots\ldots\ldots (1)$$

Head lost due to friction in pipe

$$= (750 - 680)\ \frac{144}{62{\cdot}5} = 161{\cdot}2 \text{ ft. of water}$$

$$= \frac{4\ f\ l\ v^2}{2\ g\ D}$$

$$\text{or } 161{\cdot}2 = \frac{4 \times {\cdot}008 \times 3000\ v^2}{2\ g\ D}$$

$$\frac{V^2}{D} = 108{\cdot}4 \ldots\ldots\ldots\ldots (2)$$

Multiplying (1) and the square of (2)

$$V^5 = 5279$$

$$V = 5{\cdot}55 \text{ ft. per sec.}$$

Substituting the value of V in (2),

$$\text{we get, } D = \frac{5{\cdot}55^2}{108{\cdot}4} = {\cdot}285 \text{ ft.}$$

Long Pipe Terminating in a Nozzle :—The pressure of the jet issuing from the nozzle is atmospheric and hence the whole energy will be kinetic. The pipe being long, loss of head due to friction in the nozzle is small compared with that of the pipe.

Let l = length of the pipe,
D = diameter of the pipe,
V = velocity of water in the pipe,
d = diameter of the nozzle at the outlet of the nozzle,
v = velocity of the jet issuing from nozzle.

The velocity head transmitted $= v^2/2g$.

Head lost due to friction $= (4\,fl\,V^2)/(2\,g\,D)$.

Therefore, for the maximum efficiency of transmission

$$= (4\,fl\,V^2)/(2\,g\,D)$$
$$= \tfrac{1}{2}(v^2)/2g$$

Or, $8\,fl/D = v^2/V^2$ ... (1)

H = the available head.

But $\pi D^2/4 \times v = \pi d^2/4V$

or $v/V = D^2/d^2$

or $D^4/d^4 = 8\,fl/D$ ... (from 1)

$D/d = \sqrt[4]{8\,fl/D}$ ... (2)

For any efficiency if :—

$$H = 4\,fl\,V^2/2\,g\,D + v^2/2\,g$$
$$= (v^2/2g)\left\{\frac{4\,fl\,d^4 + D^5}{D^5}\right\}$$

or, $v = \sqrt{\dfrac{2\,g\,H\,D^5}{4\,f\,l\,d^4 + D^5}}$

For the maximum efficiency of transmission :—

$\tfrac{2}{3}H = v^2/2\,g$

or, $v = \sqrt{4\,g\,H/3}$

Maximum horse-power delivered at the nozzle

$$= \tfrac{1}{3} \times \frac{62{\cdot}4 \times d^2 \times v^3}{4 \times 32 \times 555}$$

$$= \frac{62{\cdot}4 \times d^2}{256 \times 550} \times \left(\frac{4\,g\,H}{3}\right)^{\frac{3}{2}}$$

$$= {\cdot}124 d^2 H^{\frac{3}{2}}$$

Example :—The head of water at one end of a pipe 200 yards long 3 inches in diameter is 100 ft. and *f* for the main is ·01. What diameter of nozzle fitted to the end will give the maximum reaction, and what will the reaction then be? (London Univ. 1911)

Using formula (2)

$$\frac{D}{d} = \sqrt[4]{\frac{8\,fl}{D}}$$

$$=4\sqrt{\frac{8\times\cdot01\times600}{\cdot25}} = 3\cdot72.$$

$$d = \frac{\cdot25}{3\cdot72} = \cdot067 \text{ ft.}$$

$$= \cdot804 \text{ inch.}$$

1. Total head at nozzle for maximum power
$= \frac{2}{3}$ head supplied.

Reaction on nozzle = Area × Pressure

$$= \frac{\pi}{4} \times (\cdot067)^2 \times 62\cdot4 \times \frac{2}{3} \times 100$$

$$= 14\cdot76 \text{ lb.}$$

Flow in Open Channel :—In order to measure the flow of water in an open channel, Chezy formula is generally used as follows—

$$v = c\sqrt{RS}$$

where R is the hydraulic mean depth and equal to

$$\frac{\text{cross-sectional area (A)}}{\text{wetted perimeter (P)}}$$

and S is the gradient of the channel or sine of slope.

The value of c is to be found either from Ganguillet and Kutter or from Bazin.

According to Ganguillet and Kutter formula,

$$c = \frac{41\cdot66 + \frac{1\cdot811}{\eta} + \frac{\cdot00281}{S}}{1 + \left(41\cdot66 + \frac{0\cdot00281}{S}\right)\frac{\eta}{\sqrt{R}}}$$

and according to Bazin formula,

$$c = \frac{157\cdot6}{1 + \frac{r}{\sqrt{R}}}$$

The values of r and η depend on the roughness of the surface.

The following values are applicable for r and η :—

	Character of the surface.	Bazin's r.	Kutter's η.
1.	Smooth cement or planed timber	0·109	·009—.01
2.	Unplaned timber slightly tuberculated iron, ashlar and well-laid brickwork ...	0·29	·012—·013
3.	Rubble masonry and brickwork in an inferior condition : fine well-rounded gravel ...	0·833	·017—
4.	Rubble in inferior condition; canals with earthen beds in perfect condition ...	...	·020
5.	Canals with earthen beds in good condition ...	1·54	·02—·225

Neat cement or smooth pipes	·010
Smooth ashlar masonry or brickwork	·013
Ordinary brickwork	·015
Channel in earth, in bad order and regimen ...	·030
Channel and rivers with earthen beds	·035
Torrents encumbered with detritus	·050

For Bazin's r the following values can also be taken :—

(*a*) Clean smooth sides of wood, brick, stone. etc. ... K = ·29
(*b*) Dirty sides of wood, brick, stone, etc. ... K = ·5
(*c*) Sides of natural earth ... ... K = 2·35

The recent investigations indicating a more correct formula for channel flow is of the form—

$$v = kR^{x} S^{y},$$

where the indices x and y, instead of each being ·50, as in Chezy's formula, have other values.

Different investigators, such as Unwin, Thrupp, Fidler, Williams, Barnes, have determined the values of k, x and y, which are most worthy of note.

Barnes gives the following values and formulae for clean surfaces, which are the results of investigation by many other observers.

Surface.	k	x	y	Friction head h in ft.	Hydraulic mean depth R in feet.
Planed wooden flumes ...	223·3	·660	·586	$\frac{98 \times l\, v^{1.707}}{10^6 \times R^{1.126}}$	$\frac{276 \times v^{1.515}}{10^6 \times s^{.888}}$
Unplaned wooden flumes	182·5	·666	·569	$\frac{107 \times l\, v^{1.757}}{10^6 \times R^{1.171}}$	$\frac{403 \times v^{1.502}}{10^6 \times s^{.354}}$
Neat cement channels ...	136·3	·635	·484	$\frac{39 \times l\, v^{2.066}}{10^6 \times R^{1.312}}$	$\frac{435 \times v^{1.575}}{10^6 \times s^{.762}}$
Smooth-faced concrete ...	95·1	·567	·471	$\frac{63 \times l\, v^{2.123}}{10^6 \times R^{1.204}}$	$\frac{324 \times v^{1.764}}{10^6 \times s^{.831}}$
Hard, well-pointed brick	92·1	·602	·466	$\frac{61 \times l\, v^{2.146}}{10^6 \times R^{1.292}}$	$\frac{546 \times v^{1.661}}{10^6 \times s^{.774}}$
Dressed masonry channels set in cement with no projections.	109·7	·713	·483	$\frac{60 \times l\, v^{2.07}}{10^6 \times R^{1.476}}$	$\frac{138 \times v^{1.408}}{10^5 \times s^{.677}}$
Hammer-dressed dry masonry.	70·0	·820	·500	$\frac{204 \times l\, v^{2}}{10^6 \times R^{1.64}}$	$\frac{56 \times v^{1.22}}{10^4 \times s^{.61}}$
Earth canals in average condition free from vegetation.	58·4	·694	·496	$\frac{275 \times l\, v^{2.016}}{10^6 \times R^{1.399}}$	$\frac{285 \times v^{1.441}}{10^5 \times s^{.715}}$

Bends in Open Channel :—The loss of head due to friction found in the previous article is for open straight channels. Bends in channels increase the hydraulic resistances. There is no experimental data to determine the loss of head due to bends. In practice an allowance of 15 to 20 per cent. is made in the gradient for the effect of such bends with easy curves, while the conduit has moderate changes of alignment.

Most Economic Form of Channel :—The channel with most economical section is one which gives the maximum discharge for a given amount of excavation. The most economic section can be found by assuming the area to be constant when the depth of the section for maximum velocity can be determined.

(*a*) *Rectangular Channel* :—The maximum discharge for a rectangular channel occurs when the depth of water is one-half of the breadth.

Area $A = b\,h$.

Wetted perimeter $P = b + 2\,h$.

The hydraulic radius $R = \frac{b\,h}{b+2h}$.

Chezy's formula being

$$v = C\sqrt{m\,i} \text{ or } C\sqrt{R\,S} \text{ or } R = \frac{v^2}{C^2 S},$$

where m or R = hydraulic mean depth or radius,
i or S = gradient of the channel or sine of slope.

C-a coefficient covering losses due to the degree of roughness and other factors.

Substitute R, $\therefore v = C\sqrt{\frac{2\,b\,h\,S}{b+2\,h}}$.

Hence, the discharge $Q = b\,h\,v$.

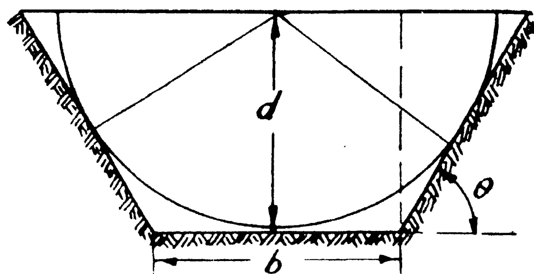
Q is maximum when $b = 2\,h$, this value is found by

making $\theta = 90°$ in $h = \sqrt{\frac{A \sin\theta}{2 - \cos\theta}}$.

(*b*) *Circular Section* :—Depth for maximum discharge $= \cdot 95 \times$ diam. Depth for maximum velocity $= \cdot 81 \times$ diameter of the channel.

(*c*) *Trapezoidal channel* :—The most economic trapezoidal section is when the three sides are tangential to a semicircle drawn on the water line with the depth of water as radius.

Fig. 4.



This condition is satisfied when :—

$d = \sqrt{A \sin \theta}$;
$b = 2d \tan \theta/2$;
$m = d/2$.

Where A = area of water cross-section ;
b = width of the channel at the base ;

$$A = \frac{h}{2}(b + b + 2h \operatorname{Cot} \theta)$$
$$= h(b + h \operatorname{Cot} \theta)$$

The wetted perimeter $P = b + \frac{2h}{\operatorname{Sin} \theta}$.

The hydraulic radius $R = \frac{P}{A}$

$$\text{or } R = \frac{h(b + h \operatorname{Cot} \theta)}{b + \frac{2h}{\operatorname{Sin} \theta}}$$

But $v = C\sqrt{RS}$

And $Q = AC\sqrt{RS}$

Q is directly proportional to $\sqrt{R}$ and naturally inversely proportional to the wetted perimeter, as $R = \frac{A}{P}$. The maximum of Q corresponds to a minimum of P.

$$b = \frac{A}{h} - h \operatorname{Cot} \theta.$$

$$P = \frac{A}{h} - h \operatorname{Cot} \theta + \frac{2h}{\operatorname{Sin} \theta}.$$

Differentiating and equating to zero

$$\frac{dP}{dh} = \frac{A}{h^2} + \frac{2 - \operatorname{Cos} \theta}{\operatorname{Sin} \theta} = 0.$$

Solving for h

$$h = \sqrt{\frac{A \operatorname{Sin} \theta}{2 - \operatorname{Cos} \theta}}.$$

$$b = \frac{A}{h} - h \operatorname{Cot} \theta.$$

θ = angle which the side slopes make with the horizontal;
m or R = hydraulic radius;
d = depth of water.

The minimum permissible slope of the side depends upon character of the soil and the value of θ must slightly be greater than the angle of repose of the material through which the cut is made.

Angle of Repose of Various Substances

Substance.	*Angle of repose in degrees.*
Sand, fine dry	31 to 37°
,, wet	26°
Vegetable earth dry	29°
,, ,, moist	45 to 50°
,, ,, very wet ...	16 to 17°
Clay dry	29°
,, damp	45°
,, wet	16°
Gravel clean	48°
,, with sand	26°
,, shingle	39°
Masonry on masonry	30°

Velocity of Flow in Open Channels :—The velocity of flow through a channel must have a permissible flow so that there may not be any tendency to erosion of the sides and bed. Ganguillet and Kutter each gives the following safe mean velocities for different materials of the channel, and they must not be exceeded

in order to prevent the washing of the bottom of the canal or its embankment :—

Material of the channel.	Safe bottom velocity in ft. per second.	Mean safe velocity in ft. per second.
Soft silty earth	·249	·33
Soft loam	·499	·66
Sand	1·000	1·32
Very fine sandy soil or loose silt	...	0·50
Light sandy soil 15 per cent. clay	...	1·20
Light sandy loam 40 per cent. clay	...	1·80 to 2
Coarse sand	...	1·50—2
Gravel	1·998	2·64
Ordinary firm soil or loam 65 per cent. clay ...	...	3·00
Stiff clay loam	...	4
Firm gravelly clay soil	5	7
Stiff clay	...	6
Pebbles	2·999	3·94
Broken stone, flint	4·003	5·57
Conglomerate	4·990	6·56
Stratified slate or rock	6·006	8·204
Hard rock	10·01	13·2
Concrete	...	15—20

Distribution of Velocities :—While water is flowing through a canal, its velocity is not the same at all points of the section. It is due to fluid friction. Due to friction of the molecules against the bank and bed the velocity of flow at the bank and bed will be less than that at the middle and thereby the discharge capacity of the channel diminishes. It is desirable to know the bottom velocity so that the soil may not be eroded.

This can be deduced from Bazin's formula,

where $V_b = v - 10{\cdot}87 \sqrt{RS}$

in which V_b = bottom velocity in feet per second.

v = average velocity in feet per second.

R = Hydraulic radius.

S = Sine of slope.

* R. G. Kennedy concludes that the safe velocity in light soils increases with the depth according to the law $V \propto d^{\cdot 64}$. For sandy soils he gives :—

Depth in feet	2	3	4	5	6	7	8
Mean velocity ft.	1·3	1·7	2·0	2·4	2·65	2·9	3·2

R = hydraulic mean depth.

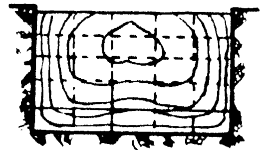
$$= \frac{\text{cross-sectional area}}{\text{wetted perimeter}}.$$

* Proc. Inst. of Civil Engineers or C. E., vol. 119, 1874-75, page 281.

S = sine of the slope of the channel with the horizontal.

From the results of various experiments Bazin has come to the conclusion that the loss of head due to friction is proportional to the wetted area and to the mean velocity. It also depends upon the nature of bed and banks, but it is independent of the pressure of the flowing water.

Fig. 5.



Distribution of velocities of water in a channel of rectangular cross-section.

Figure 5 shows the distribution of velocities of water in a channel of rectangular cross-section in which the velocities are equal at all points of a definite curve.

There are various factors such as inclination of channels, its nature, slope, character of the bed, leakage, vegetation, etc., which influence the capacity of discharge.

The growth of vegetation has the prime influence on the discharge and in many cases it reduces the discharge capacity by one-quarter to one-fifth. Specially, plants when reaching up to the surface of the water have a preponderant influence upon the flow. To some extent it increases the wetted perimeter. Hence, special care is to be given for the cleanliness of the channels. But the best practice is to increase the cross-sectional area for necessary discharge by one-fifth at the time of its construction. If the temperature is below 65°F algæ and moss growths are not serious. The growths do not take place to any extent in turpid or deep water. A mean velocity of not less than 2·5 feet per second will generally prevent a growth that would seriously decrease the carrying capacity of the canal.

Notches and Weirs:—**A notch** is an orifice with the water surface below its upper edge. It is generally used for measuring the flow of water from a vessel or reservoir. It is generally rectangular or triangular in shape.

A weir is an artificial obstruction in a water-way having a definite configuration and a sheer fall on the down-stream side over which the water falls in a sheet, as, for example, the flow of water over a dam.

The sheet of water flowing through a notch or over a weir is called the *nappe or vein*. The top of the weir over which water flows is known as *sill or crest*.

Theoretically, there is no difference between a simple rectangular weir and rectangular notch. But the *rectangular notch has sharp edges*.

Rectangular Notch :—In the case of a rectangular notch,

$$\theta = 3{\cdot}33\,LH^{\frac{3}{2}} \quad \ldots \quad (1)$$

Where, θ = quantity of water discharged in cubic feet per second.

L = breadth of the weir in feet.

H = head on the weir.

But for weirs with end contractions the above formula for discharge does not apply, for this case Francis' empirical formula is generally used, which is—

$$= 3{\cdot}33\,(L - {\cdot}1\,\eta H)\,H^{\frac{3}{2}} \quad \ldots \quad (2)$$

where η = number of contractions.

For a simple rectangular weir $\eta = 2$, and hence the Francis' formula is reduced to—

$$\theta = 3{\cdot}33\,(L - {\cdot}2\,H)\,H^{\frac{3}{2}} \quad \ldots \quad (3)$$

Another type of equation used for obtaining discharge over weir without end contraction is Bazin's formula, which is—

$$\theta = m\sqrt{2g}\;L\;H^{\frac{3}{2}}$$

$$\text{where } m = {\cdot}405 + \frac{{\cdot}00984}{H} \quad \ldots \quad (4)$$

The Bazin's formula gives the most accurate determinations with sharp-crested weirs which should be made without end contractions by lining the sides of the channel with wood sheeting or concrete for a distance back of the weir equal to about 10 H and then building the weir across from side to side.

Velocity of Approach :—Generally, the area of the channel through which the water approaches the weir, is larger than that of the weir itself. Therefore, the water will have a velocity on reaching the weir and this velocity is called the *velocity of approach.*

Let A = cross-sectional area of the channel behind the weir.

v_1 = velocity of approach.

$= \frac{\theta}{A}$ ft./second.

This velocity of approach may be assumed to be uniform over the whole weir.

The Francis' formula is then reduced to for the velocity of approach :—

$$\theta = 3{\cdot}33\,(L - {\cdot}2\,H_1)\left\{H_1^{\frac{3}{2}} - \left(\frac{v_1^2}{2g}\right)^{\frac{3}{2}}\right\}$$

where $H_1 = H + v_1^2/2g$.

From the results of the experiments Bazin found that the discharge could be obtained by increasing the actual head H by the amount $1{\cdot}6\ v_1^2/2g$.

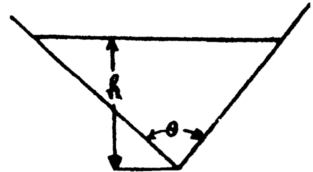
The Bazin's formula then becomes :—

$$\theta = m\sqrt{2g}\ L\ H_1^{3/2} \qquad \text{where } m = 0{\cdot}405 + \frac{{\cdot}00984}{H_1}$$

where $H_1 = H + 1{\cdot}6\ v_1^2/2g$.

Triangular Notch :—The triangular notch is the most satisfactory type in measuring the discharge.

Fig. 6.



The discharge $= 2{\cdot}56 \tan\frac{\theta}{2} H^{\frac{5}{2}}$

For 90° notch $\tan\frac{\theta}{2} = 1$.

$\therefore$ discharge $= 2{\cdot}56\ H^{\frac{5}{2}}$.

In the case of a triangular notch there is no base to cause contraction. The contractions will only be due to sides. Hence, triangular notch is used for measuring the discharge.

Precautions to be taken in weir gaugings :—For accurate measurement the following are essential :—

(*a*) Sharp-edged weir sill, fixed so as to be incapable of vibration, its face being vertical and perpendicular to the direction of flow. If rectangular, its sill should be horizontal.

(*b*) The width of weir should be long in proportion to its depth, *i.e.*, b > 3H.

(*c*) The discharge must be in air without any adherence of the vein to the weir face.

(*d*) The sectional area of vein or nappe must not be greater than one-sixth that of the channel.

(*e*) The motion of flow must be steady before reaching the weir. For this case the length of the channel should exceed 25 H. In a particular experiment of Bazin length of the supply channel was 49·2 ft., H = 1·92, ft., the maximum weir length being 6·56 ft.

(*f*) The head H must be determined accurately. The zero of the gauge should be accurately adjusted to the bend of the weir crest. The surface level is not to be taken directly over the stream but in a stilling box or pit 2 ft. square which is in communication with the stream through a pipe of about 1 inch in diameter. For accurate work the readings are to be taken with a hook-gauge provided with a vernier for reading to the nearest 0·001 ft. For automatic record the level is recorded by float operating the record pencil and such gauge must be surrounded by a stilling box in order to damp out surface oscillations.

Time required to lower the level in a reservoir by discharge over a weir.

Consider a reservoir having a horizontal sectional area A square ft. Consider it is flowing over a rectangular weir of breadth L. It is required to calculate the time required for the water level to fall from H_1 to H_2 above the level of the sill.

Suppose at any instant the height of water-level above the sill is h when a small quantity of water "dq" flows over the weir in a time "dt" thereby lowering the level by an amount "*dh*."

$$\therefore \; dq = 3{\cdot}33 \, L \, h^{\frac{3}{2}} dt.$$

But discharge from the reservoir = Adh = dq.

$$\therefore \; Adh = 3{\cdot}33 \, L \, h^{\frac{3}{2}} \, dt$$

$$\text{or,} \quad dt = \frac{A \, h^{-\frac{3}{2}}}{3{\cdot}33 \, L} \, dh$$

$$\text{or,} \quad t = \int_{H_1}^{H_2} \frac{A}{3{\cdot}33 \, L} \, h^{-\frac{3}{2}} \, dh$$

$$= \frac{2 \, A}{3{\cdot}33 \, L} \left[H_1^{-\frac{1}{2}} - H_2^{-\frac{1}{2}} \right]$$

$$= \frac{2A}{3 \cdot 33\,L}\left[\frac{1}{H_1^{\frac{1}{2}}} - \frac{1}{H_2^{\frac{1}{2}}}\right] \text{ seconds.}$$

where t = time required in seconds to lower the level from H_1 to H_2 above the level of the sill.

Considering end contractions :—

$$t = \frac{2A}{3 \cdot 33\left\{L - \cdot 2\,\frac{(H_1 + H_2)}{2}\right\}}\left\{\frac{1}{H_1^{\frac{1}{2}}} - \frac{1}{H_2^{\frac{1}{2}}}\right\} \text{seconds.}$$

Time to raise the level during flood discharge:—At the time of high flood, the weir or the spill-way cannot discharge the flood water to the same rate at which it is entering the reservoir. Therefore, the surface-level of the reservoir rises rapidly to a maximum and it falls more slowly as the flood subsides. It is important to know the rate at which the surface-level rises.

Let θ = rate of flow of the flood-water in the reservoir, in cu. ft. per second.

h = corresponding head on the weir above the level of the sill.

If the weir be rectangular, the excess of in-flow over out-flow will be—

$$\left(\theta - 3 \cdot 33\,L\,h^{\frac{3}{2}}\right) \text{ cu. ft./second.}$$

If A be the horizontal sectional area of the reservoir and dh be the rise of water-level in time dt,

$$Adh = \left(\theta - 3 \cdot 33\,L\,h^{\frac{3}{2}}\right) dt$$

$$\text{or, } t = \int_{H_1}^{H_2} \frac{dh}{\theta - 3 \cdot 33\,L\,h^{\frac{3}{2}}}$$

where t is the time required in seconds to raise the level from H_1 to H_2.

To integrate the above expression put—

$$\theta = 3 \cdot 33\,L\,H^{\frac{3}{2}} = \text{Constant.}$$

Also put $h = Hr$

$\therefore dh = Hdr.$

$$\therefore t = \int \frac{Hdr}{3 \cdot 33\,L\,H^{\frac{3}{2}}\left(1 - r^{\frac{3}{2}}\right)}$$

$$= \frac{H}{3{\cdot}33\ L\ H^{\frac{1}{2}}} \int \frac{dr}{\left(1 - r^{\frac{3}{2}}\right)}$$

for the integral portion put $r = \sec^2 \theta$

$\therefore\ dr = 2 \sec^2 \theta \tan \theta$

$$\therefore \int \frac{d\,r}{(1 - r^{\frac{3}{2}})} = \int \frac{2 \sec^2 \theta \tan \theta\, d\, \theta}{1 - \sec^3 \theta}$$

Again put $\sec \theta = k$

$\therefore\ \sec \theta \tan \theta \times d\,\theta = d\,k$

$\therefore$ integral is reduced to—

$$\int \frac{2k\, d\,k}{1 - k^3}$$

$$= \frac{2}{3} \left[\tfrac{1}{2} \log (1 + k + k^2) - \log (1 - k) - 2\sqrt{3} \tan^{-1} \frac{2\,k + 1}{\sqrt{3}} \right]$$

$$= \frac{2}{3} \left[\log \frac{\sqrt{1 + r + \sqrt{r}}}{1 - \sqrt{r}} - 2\sqrt{3} \tan^{-1} \frac{2\sqrt{r} + 1}{\sqrt{3}} \right]$$

as $k = \sqrt{r}$

$$\therefore\ t = \frac{2}{3 \times 3{\cdot}33\ L\ H^{\frac{1}{2}}} \left[\log \frac{\sqrt{1 + r + \sqrt{r}}}{1 - \sqrt{r}} - 2\sqrt{3} \tan^{-1} \frac{2\sqrt{r} + 1}{\sqrt{3}} \right]$$

$$\therefore t = \frac{2}{3 \times 3{\cdot}33\ L\ H^{\frac{1}{2}}} \left[\log \frac{\sqrt{1 + \frac{h}{H} + \sqrt{\frac{h}{H}}}}{1 - \sqrt{\frac{h}{H}}} - 2\sqrt{3} \tan^{-1} \frac{2\sqrt{\frac{h}{H}} + 1}{\sqrt{3}} \right]_{H_1}^{H_2}$$

Construction of Weir :—For large weir it is better to make them of concrete using a thin steel inset to give sharp edges around the notch. Like a concrete dam the weir must be self-supporting.

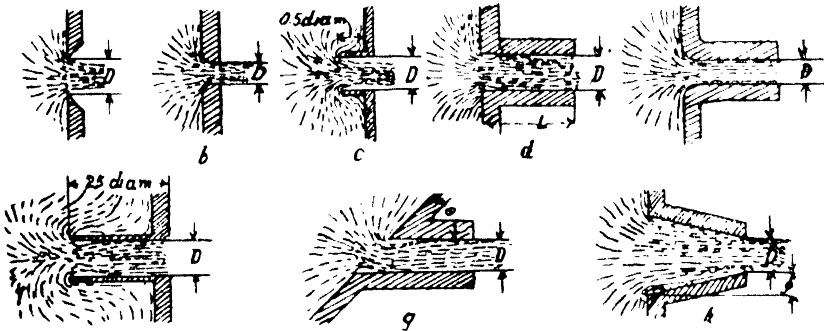
Small weirs may be constructed of planks and timber. It should have a floor on the down-stream side to prevent erosion of the stream bed from the over-fall. The weir must be vertical and in order to prevent settling of one side or the other, heavy

stones should be sunk under the mud sill. Triangular sharp-edged weir gives the very accurate result in the measurement of discharge.

Orifices

The various types of sections of orifices are shown in the figures below :—

Fig. 7.



Of the various types of flow of water through the orifices the following types are of importance :—

(1) Efflux through sluice gates.
(2) Flow through penstock intakes.
(3) Efflux through nozzles or guide vanes of water wheels.

For Small Orifices

The general equation for the velocity of spouting water is $V = \sqrt{2gh}$,

where v = velocity of flow in feet/sec. through the orifice.
h = head of water above the orifice.

Considering friction, actually $V = C_V \sqrt{2gh}$
Where C_V = coefficient of velocity

$$= \frac{\text{actual velocity}}{\text{theoretical velocity}}$$

The discharge through the orifice is effected due to the following facts :—

(1) Area of jet at the vena contracta is less than that of the area of the orifice.

(2) Actual velocity at the vena contracta is less than the theoretical velocity.

$\therefore \theta = A \times C_c \times C_V \sqrt{2gh}$

Where C_c = coefficient of contraction and
C_d = coefficient of discharge.
A = area of the orifice.
$\therefore C_d = C_c \times C_v$.

		Value of C_d.
(*a*) Sharp-edged orifice	...	·61
(*b*) Rounded orifice	...	·97
(*c*) Inwardly projected orifice	...	·5
(*d*) Short tubes with sharp-cornered entrances		
for vaules of L/D = 0	...	·60
,, ,, ,, = ·25	...	·63
,, ,, ,, = 1	...	·76
,, ,, ,, = 1·5	...	·79
,, ,, ,, = 2·5	...	·80
,, ,, ,, = 3·5	...	·80

			Value of C_d.
(*e*) Short tube with rounded entrance		...	.97
(*f*) Inwardly projecting tube with sharp-cornered entrance	...	... ...	·72 to ·8
(*g*) Inclined short tube with sharp-cornered entrance.			
For values of L = 90°	...	...	·82
,, ,, = 80°	...	...	·80
,, ,, = 60°	...	...	·76
,, ,, = 50°	...	...	·75
,, ,, = 30°	...	...	·72
(*h*) Convergent short tube sharp-cornered entrance.			
For values of θ = 0°	...	...	·82
,, ,, = 5·75°	...	...	·94
,, ,, = 11·25°	...	...	·92
,, ,, = 22·50°	...	...	·85

For Sluice Gates :—

Let h_1 = head of water on the upper edge of the opening.
h_2 = head of water on the lower edge of the opening.
b = width of the opening.

Consider h the head on any horizontal element of the orifice of thickness d h.

$$\therefore \theta = \int_{h_1}^{h_2} b \times dh \times cd(h_2^{\frac{1}{2}} - h_1^{\frac{1}{2}})$$

$$= 2\sqrt{g}\, b \times C d \left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right)$$

$$= \tfrac{2}{3} \times 8{\cdot}02\, b \left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right)$$

$$= 5{\cdot}347\ b\ \left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right)$$

Actually, $\theta = 5{\cdot}847\ C\, b\, \left(h_2^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right)$
where C is a coefficient having different values for different gate areas.

Barton (Punjab Irrigation Branch Bulletin) gives the following values of C :—

Width of gate in feet =	4 —	6 —	8 —	10
C =	·748 —	·763 —	·779 —	·795

The value of C becomes more for larger areas of the orifice.

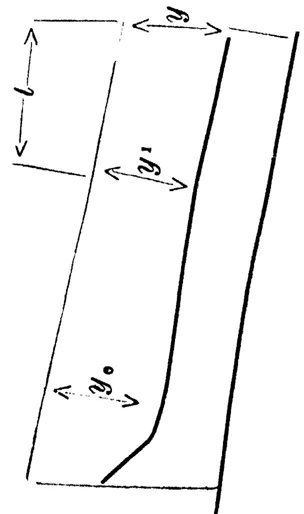
Efflux from Nozzles :—A nozzle having a smooth, tapered interior cross-section, will have a coefficient of discharge equal to nearly unity. The average coefficient is 0·99.

Varied flow :—So far we have considered a uniform flow, when the channel has a constant slope and uniform cross-section. But if such conditions are not available, the flow will be varied.

The expression of varied flow will be as follows :—

If l, y_1 and y_2 are as shown in the Fig. 8.

Fig. No. 8.



v_1 = Mean velocity at y_1,

v_2 = Mean velocity at y_2,

P = Wetted perimeter,

A = Area of cross-section,

f (v) = v in Kutter's formula,

a = 1·10

Q = Quantity of discharge, being given the longitudinal profile and several transversal cross-section,

$$y_2 - y_1 = \frac{v_2^2 - v_1^2}{2g} + \int \frac{P}{A}\, f(v)\, dl$$

approximately,

$$or,\ y_2 - y_1 = a\left[\frac{v_2^2 - v_1^2}{2}\right] + \int \frac{P}{A} f(v)\, dl$$

$$\text{and } Q^2 = \frac{y_2 - y_1}{\frac{a}{2g}\left[\frac{1}{A_2^2} - \frac{1}{A_1^2}\right] + \frac{cl}{2}\left[\frac{P_1}{A_1^3} + \frac{P_2}{A_2^3}\right]}$$

Where A_1 = Area of cross-section at y_1,
A_2 = ,, ,, ,, at y_2,
P_1 = Wetted perimeter at y_1,
P_2 = ,, ,, at y_2,
C = Coefficient as given by Bazin

Bazin's coefficient C has the following values :—

$$C = \frac{1}{\sqrt{0{\cdot}0000045\ (10{\cdot}16 + \frac{1}{R})}},$$ for very even surfaces, fine plastered sides and beds, planed planks.

$$C = \frac{1}{\sqrt{0{\cdot}0000013\ (4{\cdot}345 + \frac{1}{R})}},$$ for cut stone, brick-work, mortar, unplaned timber.

$$C = \frac{1}{\sqrt{0{\cdot}00006\ (1{\cdot}219 + \frac{1}{R})}},$$ for slightly uneven surfaces such as rubble masonry.

$$C = \frac{1}{\sqrt{0{\cdot}00035\ (0{\cdot}2438 + \frac{1}{R})}},$$ for uneven surfaces such as earth.

P. 58, Hydro-Electrical Engineering, by Richard Muller.

CHAPTER VII

ECONOMICAL DEVELOPMENT OF WATER-POWER

Capacity of Development.—A water-power to be *economically developed* must practically be doveloped to certain proportions regardless of its market. The economic and efficient considerations of the development of a hydro-electric scheme is most important for the designer. Powers on large rivers can seldom be developed and operated successfully in a small way. On a given river it is often as expensive to develop a small amount of power as to develop the stream to its full capacity. The same dam with the same appurtenances is usually necessary whatever the developments. The same operating force will usually be required whether the plant is fully or partially loaded and whether the development is partial or complete. If the power is transmitted, the same towers required for 10000 kW. will carry 20000 kW. successfully. A completed full development will involve a larger power house, a few more turbines, generators and equipment and a little larger transmission wire. These are usually the only extra expenses involved. Hence, on large streams the projects must be sufficiently large to pay only when an adequate load is secured. The investment in these plants is so great that they can never be built except for an existing market which will provide at least their principal load unless industries are developed in connection with them. Both fixed charges and operating cost begin at once when the plant is constructed, and interest starts with construction. A market must be obtained immediately on completion in order to meet fixed expenses or the plant must go into bankruptcy.

Useless Source of Water-Power

In spite of great importance of water-power, many of the falls in existence may prove economically useless, either on account of their being only small and fluctuating rise and fall in the water distance from centres of industry, the lack of transport facilities or from the fact that it is prohibitive in cost or that a costly storage system is necessary to give a sufficiently continuous supply. If water were the only source of power, the outlook would be different; but it is in active competition with other agents such as coal and oil,

The cost of hydro-electric power is mainly made up of charges against capital, interest, depreciation, sinking fund charges, taxes and insurance charges, etc. These charges are always much greater than the water charges and the cost of operation, maintenance and supplies. The local circumstances and the physical characteristics of the site determine, to a great extent, the capital charges. Where the available head is great and the storage provided by a natural lake or where the storage is unnecessary, they may be comparatively small. If, on the other hand, extensive works are required (1) to store the water and (2) to bring it to the power-house, the over-all cost of power may be largely in excess of that generated by the steam plant; and (3) the cost of water rights and (4) that of transmission and distribution are heavy. The capital charge will be prohibitive and such scheme will not be worth attempting.

As already stated, there may be a natural fall or a fall may be artificially developed. The same fall varies at different parts of the year, but the load remains constant or the load may also vary with the season. But the variations in natural stream flow correspond in no degree to the variations in power demand and many factors are to be taken into consideration for determining the most economical capacity of the hydro-electric plant and of the storage and pondage that must be created to utilize to the greatest economical extent, the flow of the stream. The factors are interrelated and it is no easy matter to solve some problems.

The most important points to consider are :—

(1) The market requirements.

(2) The available head at the site.

(3) The natural flow of the stream.

(4) The probable cost of development.

(5) The value of the output.

(6) The nature and capacity of existing and proposed auxiliary power plants.

(7) The extent to which the plant is to be installed to carry peaks of inadequately ponded plants.

(8) The extent of possible economical regulation of the flow.

(9) The class of power to be produced, *i.e.*, (1) primary power, or (2) secondary power.

(1) **Primary Power**:—If the power to be supplied is to be uninterrupted and there is no storage, pondage or auxiliary plant, the capacity of the development must be limited so as to correspond to the minimum natural flow of the stream.

If the installation is to serve primary power, it must be uninterrupted and it must be supplied in preference to other demands for power as in town lighting, as the supply is generally guaranteed to the consumer. There must be special provision to supply this power, when other demands for power may not be met, such as during the repair of a canal or a break-down of the base plant.

(2) If the development is to serve a **secondary-power market**, a part of the installation will be idle during periods of low water. At such times there is ample capacity, and during periods of abundant flow, a shut-down of a unit affects secondary power only. Therefore, a spare unit is not necessary in such cases.

If there is adequate pondage to be provided, the capacity must be limited to an amount such that the average output corresponds to the minimum flow of the stream.

If only storage is available, but no pondage, the capacity must be limited to correspond to the minimum regulated flow of the stream.

If both storage and pondage are to be provided, the capacity must be limited to an amount such that the average output corresponds to the minimum regulated flow. In all cases the capacity of the development may be increased to some extent.

The probability of deposits of silt destroying the effectiveness of both pondage and storage reservoirs should be carefully investigated, as excessive silting has frequently proved serious, and no effective means for removing enough of the silt to restore the usefulness of the reservoirs has yet been found. If a number of storage reservoir sites are available, sufficient funds should be set aside from the earnings of the plant to build new reservoirs to keep pace with reduction of available storage capacity due to silting.

The Number of Units :—A few large units, as compared with a number of smaller units, is a distinct advantage as far as the cost of the development is concerned, because the cost per horsepower decreases as the size of the unit increases. The required number of units depends upon the nature of the load and the number of plants in the system. A development that is a part of an extremely large system can well afford to have only two units or even a single unit, because a unit out of commission in such cases would mean a very small percentage of total power temporarily lost and, with a very large system, there is usually a steam auxiliary or other means of supplying the deficiency.

On the other hand, a development that alone serves a market cannot afford to rely on a single unit, for obvious reasons. Such development, if serving an important market without a steam plant

stand-by, must have a spare unit for emergency; and the spare unit would, of course, be of the same size as the other unit. To provide a spare unit for a single-unit plant doubles the installation. A spare unit for a two-unit plant adds 50 per cent. and, for a three-unit plant, adds $33\frac{1}{3}$ per cent. Consequently, if a spare unit is required, the total installed capacity decreases as the number of units increases; but, since the cost per horse-power increases as the number of units increases, there is for each case a definite and readily determined number of units that is most economical. The choice of units also depends upon the nature of demand.

Types of development:—The development consists of a main dam, a diversion weir, a conduit, a forebay.

The development may be:—(1) Direct—In this case all the available fall at the dam may be utilized, the power station is located at its end or in the interior of the spill-way. As a rule, this kind of development is adaptable only to central power stations where only one or two plants are to be installed.

Advantages:—

(1) The entire plant is concentrated at one point.

(2) Saving in the operation cost.

(3) The highest obtainable hydraulic efficiency of the power components, fall and flow.

Thus, while the direct development plan realises the highest percentage of flow and fall, and represents the greatest simplicity of works and lowest operating charges, and, therefore, as a rule, the most economical, the conditions may sometimes be such that its adoption is prohibited by the first cost or by considerations of safety and of continuity of operations.

The selection depends upon a comparison of :—

(1) cost of the dam and embankment and cost of a lower dam and diversion works,

(2) cost of flowage for the upper pool,

(3) advantages secured by an extensive pond area,

(4) the flood flow conditions as affecting the power house, the rise in the lower pool and fluctuations in the working head.

Classification of Types of Development

Vide Figs. 6, 7, 8, 9, 10 in pages 74 and 75 and Figs. 21-24 in page 90.

(1) *Diversion type with dam*:—The fall is developed by means of a dam as in the 1st case, but the water is distributed to one or more plants by means of a long head-race canal through

which the water flows to the power station, after which it is discharged either into the stream in some part below the dam or into a tail race from which it is finally discharged at a point lower down the stream.

This type may be of two kinds :—(*a*) The short development ; (*b*) the distant diversion.

(*a*) *The short development* :—This is suitable in places where the dam location of the power house is not feasible because of contraction of the river channel or of the insufficient height of the spill-way to accommodate the power equipment in its interior. The power house is then located as close below the dam as practicable, but at a safe distance from the spill-way overfall. Water is conducted from the spill-way pond in accordance with the volume to be utilized, in a canal, flume, or pipe.

(*b*) *The distant diversion* : —It is very good when the concentration of the available fall at one point is not feasible or is too costly. The spill-way or reservoir dam is located at the most advantageous point and the water is conducted from there to the lower level by a canal, flume or pipe line and the power station is at the terminal. The features of this class are very similar to those of the short diversion programme, the difference being only the distance of diversion.

Diversion of works :—This forms a part of the plant only when the power station is located some distance downstream from the dam, and then consists of an intake, the conduit, which may be an open canal, flume or pipe diverting the water from the upper pool, and the forebay.

(2) *Diversion with or without dam* :—The development may be with or without a dam at the head of the rapids or fall which is to be utilized and the water is conducted through a long head race, if land of a suitable elevation is available or otherwise through a tunnel or elevated flume to a point immediately above the site of the power station. From the end of the head race, tunnel or flume the water is carried to the plant through a pipe line.

(3) This is like the last, but the ground being unfavourable, where the head race or tunnel is used a long pipe line is provided to conduct the water from the head works to the station.

(4) This is the tunnel tail race type and it involves conducting the water through pipe line directly to the wheels, from which water is discharged into a tunnel tail race through which the water is discharged back into the stream.

Storage and Pondage

The demands for power ordinarily correspond in no degree to the varying natural flow of the stream. Hence, storage and pondage is very often necessary to regulate the flow to correspond with the power demand. Therefore, unless a large part of the natural flow is to be wasted, storage and pondage must be provided to regulate the flow so that it may be made available at times when the generation of power is required.

The word "**storage**" is used to indicate the building up of the low natural flow of the stream to a uniform discharge. "**Pondage**" is used to indicate the regulation of the resulting uniform flow, or the natural flow, if there is no storage, to suit variations in weekly load demand. If storage is at the site of the development, it will also provide the necessary pondage.

Storage consists in the impounding and accumulating of considerable quantity of the excess run-off during seasons of surplus flow for use during dry seasons. Ordinarily, the contents of storage reservoirs are led out in such a manner that, when they are added to the natural run-off from the intermediate area between the reservoir and the site of the development, the resulting flow at the development will correspond as closely as possible to the flow demanded for power. As regards storage of water in the earth percolation and capillarity are of special consideration in that they bear a relation to the important factor of run-off. Storage is one of the important subjects of the conservation of natural resources.

When water is impounded in this way, it has additional value and high over-all efficiency of the waterways. The judicious use of such water should be a very valuable and important phase of the operation of such systems.

In practically all countries there occurs a season with subsequent waste of water and in order to make full use of the run-off, it is necessary to use storage and regulate its flow.

Its scope is the diminution of floods, the decrease of destruction to life and property caused by them, the improvement of river navigability and a utilization of a part of these large and now wasting, and destruction carrying flood waters in and for power development and for irrigation also. Hence, the three factors of reservoirs are :—(1) inflow or filling, (2) holding, and (3) outflow or using of supply of water as required. If it is not feasible to obtain reservoirs of sufficient capacity to effect complete regulation, partial regulation is often found advantageous.

Advantage:—The benefit derived from storage is measured by the resulting decrease in energy required from the auxiliaries to maintain continuous output and by reduction in peak demand of auxiliaries or in the case of an independent water-power by the resulting increase in both the energy output and the value of such output, or by the extent to which the water-power is made continuous and reliable.

Regulated flow from reservoirs is the principal agent for mitigating flood and low water extremes of stream.

Thus storage is of the greatest importance in hydro-electric development, as it will usually add greatly to the earning capacity of any such development.

This fact may be well illustrated from the Cauvery power scheme.* "Since the initial installation of 6000 horse-power generating plant, there had taken place six further installations bringing the capacity of the power plant at Siva Samudram to 50000 E. H. P.

The initial installation was a development of the minimum stream flow in the river. The generating capacity was happily sufficient to supply the then-estimated power requirements of the Gold Mining Companies at Kolwar. The generating capacity was increased to 13000 horse-power in two additional stages and incorporated such minor storage features as were possible by temporary water conservancy measures. During this period, the growth of general power and lighting load in the cities of Bangalore and Mysore altered the problem of power supply. The power had thereafter to be transmitted to a large number of consumers and no failure of power supply on account of river water shortage could be permitted.

A storage feature had, therefore, to be included in the scheme. Accordingly, the Krishna Raja Sagar reservoir was built for the purpose of affording the necessary storage. Further, additions to the generating plant were then made with the confidence gained of an assumed water supply in the river consequent on the construction of the storage reservoir."

The cost of the dam is Rs. 250 Lakhs.—

Maximum flood discharge in the river ...	240000 cusecs.
Maximum discharge provided for ...	350000 cusecs.
Horse-power developed at Shiva prior to the construction of the said dam ...	10000 H. P.
Increase of out-put with the help of the storage in dam	36000 H. P.

* Page 146, History of the Cauvery Power Scheme, by M. Gopal Krishnaiya.

This reservoir can supply water sufficient for the present generation for 3 years in case of continuous drought.

This remarkable increase in output would not have been possible if the Krishna Raja Sagar Dam had not been constructed.

Disadvantages :—The disadvantage is that sufficient impounding cannot be maintained to give any stream the power representing its maximum flow and the submersion of valuable land.

Classification of reservoirs :—Reservoirs may be of two types—(1) natural lakes or (2) artificial ponds. The former may only consist of a dam and the outlet or embankments, or dikes may be made across some low defiles. Such lakes may have more than one natural outflow which are closed when they seem as reservoir. Artificial reservoirs are generally in the valley of some tributary or on the head water streams and are created by means of a reservoir dam which is provided with a sluice way.

Another classification is based upon the function of the reservoirs (1) **Regulating reservoirs** containing a few hours' supply constructed close to the forebay or as near the ground as possible. The foregoing considerations then apply to a great extent. The reservoir may either be on a higher level, connected to the forebay by pipe or open conduit, or (if close) may be on the same level and connected by a closed pipe ; the latter arrangement is preferable, as the two storages then become virtually one, whereas in the former design the outflow of the reservoir has to be regulated to meet all changes of load. In either case, it should be possible to isolate a small chamber in the reservoir, where the supply to the forebay is taken off and, while the open channel (when there is one) should enter the reservoir as far away as possible from this point, there should be an emergency bye-pass directly into it for use during the repair or cleaning-out of the reservoir. The reservoir, in conjunction with the forebay, will keep the plant working in case of a temporary breakdown in the open channel or headworks. In addition, it will (as already explained) store the day's supply for use at times of heavy load.

As in the case of the forebay, a bed slope and large *scour valves are* required, since no silt traps higher up will prevent fine mud being carried through to deposit in the still water of the reservoir. An *escape sill* is also necessary to discharge any overflow safely.

Most regulating reservoirs are artificial structures of masonry or concrete built in excavated ground like the forebay. Expense, therefore, puts a limit to their practical capacity and

economic depth, while the ground often limits their superficial area. If the water occasionally carries a heavy burden of silt, it is highly advisable to sectionalize the reservoir from the commencement so that one division can always be available. This is well illustrated in the Nainital intake chamber in a small way. To do this subsequently is a difficult matter. It is necessary to even up the level of the natural channel and divert the water to where it will be tapped off into a canal or directly into a forebay. In canals, the headworks may consist of a diversion channel at a point where a fall occurs; and the same may be said of falls on rivers where it may happen that very little in the way of permanent works is required, if the ground is naturally suitable. Gratings are, however, always necessary to prevent timber, ice, or general debris from entering into the operative part of the system; and controlling gates are needed for regulating the supply and for shutting it off in case of accident, or for repairs or cleaning-out operations. It is also often necessary to carry out a certain amount of river training work to ensure the safety of the headworks proper.

(2) **Main reservoir** which stores water for weeks or months is created by means of embankments or dams. These larger storages, of course, are also regulators; but generally they are distant from the forebay, and the daily water regulation is preferably carried out there.

The important points for consideration are:—(1) The configuration of the ground on the geological formation which must be suitable; (2) the value of the submerged land which is utilized for the natural reservoir sites, this land may, on the one hand, be barren, waste, or, on the other, highly cultivated land containing villages, the inhabitants of which must be displaced and provided for elsewhere. At Shiva the submersion is—

(1) Irrigated area submerged = 11934 acres.

(2) Dry cultivation submerged = 13934 acres.

(3) Villages submerged = 25 villages.

Nevertheless, storage on a moderate or even on a small scale may be invaluable as an adjunct to a perennial stream. For whereas power is for the most part only required for some 10 or 12 hours a day, and the requirements fluctuate during this working day, the stream will flow on; so if the unutilized flow during the 12 or more idle hours can be stored, it will allow at least double the draw-off during the working hours, and will, therefore, double the amount of power.

It is generally deepened and widened river channel, that the water spreads over the otherwise normally dry river bottom up into various tributaries and into low marshes and meadows. It is not confined by artificial embankments unless it is necessary at certain points to prevent its spreading over high ways or more controlled lands.

Reservoirs may vary from mere regulators containing a few hours' supply up to lakes holding six months' rainfall or more. Generally speaking, low-fall plants are fed from perennial rivers or canals with a large and fairly constant supply, and it is only necessary to ensure enough regulating storage to deal with the hour-by-hour or day-by-day fluctuations of load; anything more than this would generally be prohibitive in cost. As the falls to be dealt with become larger, and hill streams come into play, the flow becomes more irregular, and it may be necessary to store enough water to tide over long droughts by means of considerable reservoirs. The limiting case is where there is practically no flow except during the short monsoon period of heavy rainfall, the whole available run-off of the catchment area being then impounded for use during the long dry season. Compare the Tata Hydro and Andhra Valley schemes. Small reservoirs may be purely artificial constructions of masonry or concrete, built in excavated ground; while limitations of cost confine the more extensive lakes to such as can be constructed by means of earthen or masonry dams.

Pondage is not unmixed good as it can only be utilized at the expense of operating head. This disadvantage can, however, be counteracted by providing temporary flash-boards by which the normal level may be raised several feet—*Vide* Fig. 20, p. 89.

(3) **Location and general consideration for reservoirs** :—The reservoir site should be looked for along tributaries above the power site and lakes and swamps in the drainage area. The location of the site in relation to the drainage area and with respect to its point of distribution should be so as to make the cost of outlets and conduits minimum.

First determine the quantity of water which the reservoir may hold. Allow a certain dead space at the bottom of the reservoir. Consider the silt and mud that may be deposited on the bottom. Provision must be made for outlets at the bottom of the reservoir so that excess accumulation of silt and mud may be sluiced away.

Outlets :—If the reservoir is in connection with low heads, the outlets from the reservoirs may be forms of slide gates in

excess of 50 feet which may be used under heads. If the head is high, difficulty is due to (1) close regulation, which requires that the movable port of the valve be constantly in contact with the highest velocity of the water.

Gates and valves are admirable for outlets of large reservoirs. Consider also seepage and evaporation and carefully insure imperviousness of the reservoir bottom. Sometimes it is necessary to trip the top soil until impervious strata are reached while fissures may have to be closed.

If the available storage is insufficient to regulate the stream so as to provide the flow demand, continuously during years of low run-off, the reservoir may be operated to provide maximum possible total energy output or to provide maximum possible primary energy output. Special attention should be given to the reservoir outlet, as conditions of foundation material for dam, character of banks, height and slope of banks, section across the stream, etc., may affect the type of and cost of dam, spill-way and complete construction.

Capacity of reservoirs :—In many cases it must be such as to insure reasonably uninterrupted demanded flow. Failure to fulfil primary power contracts is objectionable from a business stand-point. There may be a shortage of water supply in 10 or 20 years causing interruptions of power supply. It may be preferable to the expenditure necessary for larger storage. For more important markets, however, the storage should be of such capacity that the deficiency may not be possible except on an average of once in 50 or 100 years.

Considerations which determine the amount of storage :—

(1) Dependibility of water supply.
(2) Topography with reference to the angle of repose of the ground, etc.
(3) The geology with reference to the character of bed and banks of the reservoir.
(4) The effective depth of reservoir.
(5) The locality with respect to the development as a whole.
(6) The character and the cost of the dam or wall.
(7) The cost of inundated land.

V. **Storage Calculations for Reservoirs** :—In many cases it is required to determine the reservoir capacity which will insure reasonably uninterrupted demanded flow. The reservoir is to collect water in flood days which can be utilized for power purposes when the in-flow is less than its out-flow, *i.e.*, if in any month the average in-flow is less than the average flow demanded. As the mass curve or the flow-curve has been drawn for total

run-off throughout the year after considering seepage and evaporation, the capacity for the required storage for the year can be calculated from them.

Procedure to Construct a Reservoir:—Make a reservoir survey—contours of from 4 to 10 feet intervals or 5 feet contours are taken; very large storage areas have verticals taken to 10 ft. The survey must be accurate within range of 8 or 16 per cent. The contours should be extended above the highest elevation to which there is any possibility of taking the head water. Water levels and estimated height of back water should be shown on the contour map. The profile should also contain section through the dam, intake, etc., and should show the governing features of elevation, such as head water crest of dam and other necessary information.

In order to calculate the area of storage basin it should be considered that it is not feasible to lower the lake level to a point below which the reduction in head is 25% of the normal head.

Hence, the upper layer of water in the lake having a thickness equal to 25% of the normal head is that useful for power production.

The formula for the calculation of the area of a storage basin is as follows:—

$$Q = \frac{A' + A''}{2} \times \frac{H}{4}$$

where Q = volume of water to be stored in good times for power.
A′ = area of the lake at normal elevation.
A″ = area of the lake at elevation = 75% of the normal head.
H = head in feet with water at normal elevation.

The value of A″ is generally 75 per cent. of normal head.

In case of extreme necessity a drop of head more than 25% may be allowed, but all calculations as to the amount of power obtainable from a given stream with storage should be based on a drop in head not exceeding 25 per cent.

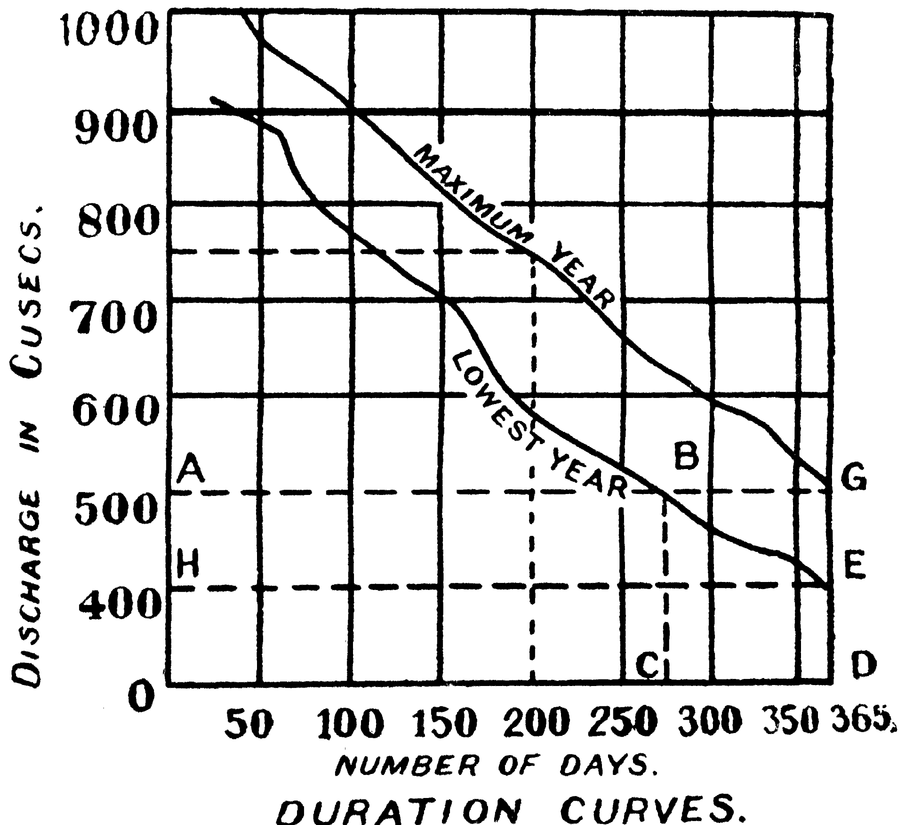
The amount of storage is more often regulated by the financial considerations than engineering possibilities. The storage area will be less, if the head of water is high.

Anyhow it is required to determine the reservoir capacity which will insure reasonably uninterrupted demanded flow. Failure to fulfil primary contracts is objectionable from a business stand-point, even though penalties and damage suits do not result.

Duration Curve

This provides another convenient means of studying the economic development of a stream. It is obtained from a record of discharge of the stream extending over a long period by plotting the discharge as ordinates against the number of days during the period of record (or a part only of the period say a low year *i. e.* a year in which the average discharge of the stream was lowest as compared with any other year), for which the discharge was equal to a given definite value as abscissa. For

Fig. 1.



DURATION CURVES.

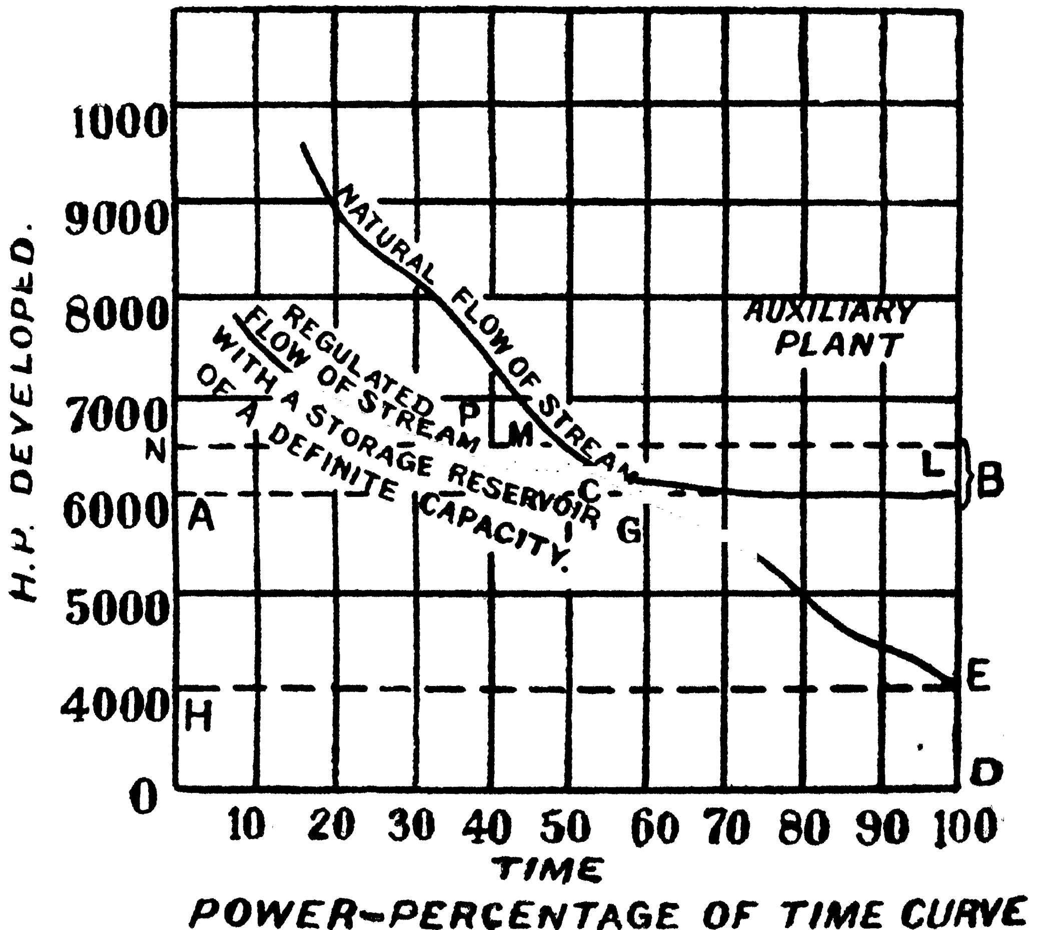
example in Fig. (1) which shows the "DURATION CURVE" of a stream for the year of maximum average discharge, and that for a year of minimum average discharge, then in the latter case a discharge of say 500 cusecs was obtained for some 270 days.

Similarly for the year of maximum average discharge a flow of about 740 cusces was available for 200 days.

For a given head, a given amount of discharge represents a definite amount of power; by representing the number of days for which a given definite discharge was available as a percentage of the total time and plotting the power developed against the percentage of the total time for which that power was available, we get the Power-Percentage of time curves which is the same thing as the Duration Curve with the new *co-ordinates*.

Thus, taking the minimum year as the basis of computations, suppose a hydraulic plant sufficient to utilize a discharge of 500 cusecs is installed; then the total number of power units

Fig. 2.



POWER-PERCENTAGE OF TIME CURVE

generated by the natural flow of the stream will be represented by the area $\overline{OABED}$. Of this, the amount of power available continuously throughout the whole time *i. e.*, 100% of the time will be that corresponding to the minimum discharge $\overline{DE}$ of the stream equal to 400 cusecs and the number of primary power units thus available will be represented by the area $\overline{OHED}$. The remainder power units represented by the area $\overline{HABE}$ can only be sold as secondary power which brings lower revenue per unit than primary power. If now storage be provided so that we can generate power units represented by the area $\overline{BGE}$ from the stored water, then the whole of the secondary power units $\overline{HABE}$ becomes converted into primary, thus bringing more revenue.

With a head of 110 ft. and turbine efficiency of 80 %, a discharge of 400 cusecs will give us $\dfrac{400 \times 62{\cdot}5 \times 110 \times {\cdot}8}{550} = 4000$ H. P.

By the help of "Mass Curve" we can find out the size of storage reservoir to satisfy a given demand, say, corresponding to 6000 H. P. continuously, as well as the percentage of time for which a greater demand may be met with by the same reservoir. Plotting it up suppose we find the conditions as in Fig. 2, then if $\overline{DB}$ = 6000 H. P. be the primary power supplied without the help of any auxiliary plant, the number of units to be generated from D stored water will be given by the area $\overline{GBE}$.

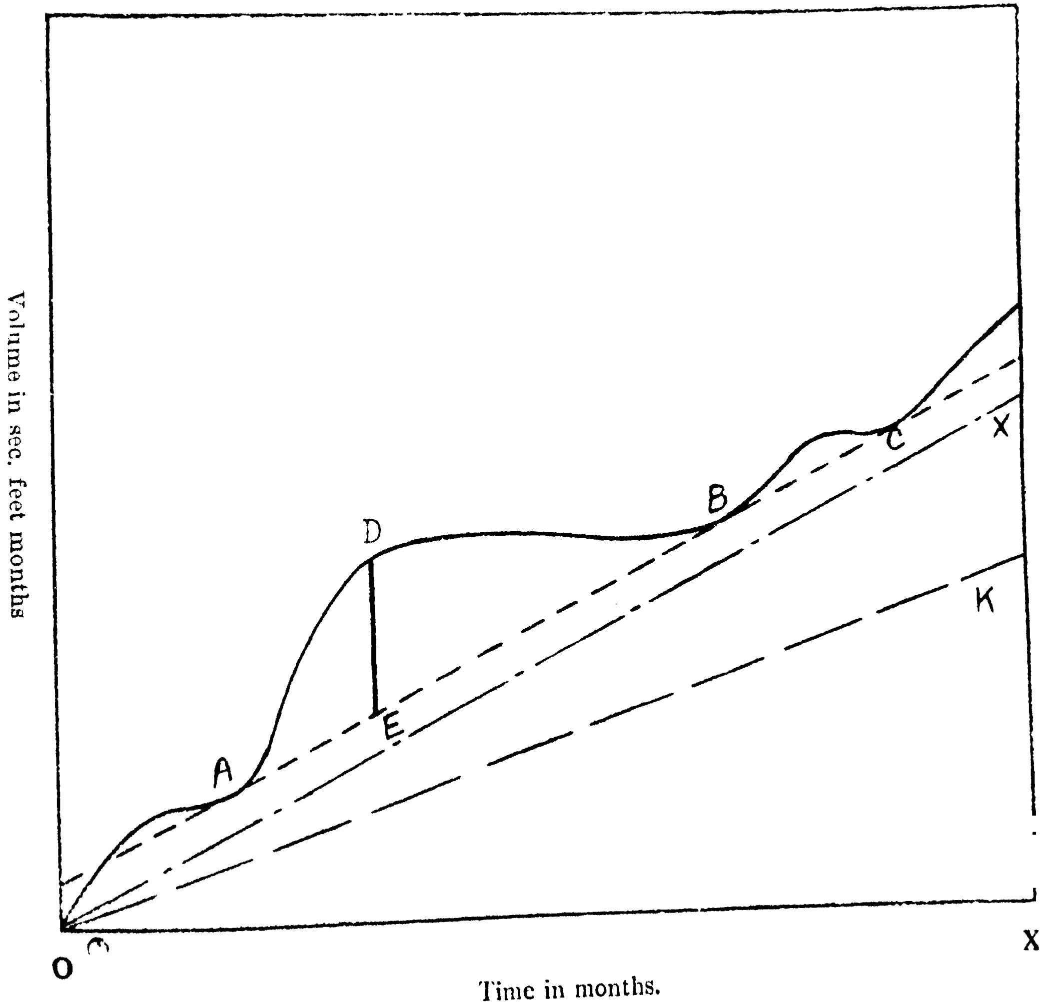
If now auxiliary plant of capacity $\overline{BL}$ = 500 H.P. be installed to bring up the continuous minimum capacity of station to 6500 H. P., the number of units to be generated by the auxiliary plant will be represented by the area $\overline{PCBL}$ and it will further convert an amount of secondary power represented by the area $\overline{NPCGA}$ into primary power.

It will be realised that the relative advantage of installing an auxiliary plant becomes smaller the more a stream is developed by storage.

The Mass Curve

In many cases it is required to equalize the flow of a stream over a long period. In such cases the necessary reservoir capacity may be determined graphically by drawing a "mass curve." For this net monthly run-offs in second-feet months are to be added after correcting for evaporation and seepage. A curve is to be plotted with the total run-off as the ordinates on a time base. The slope of the curve at any point represents the rate of flow at that instant.

Fig. 1.



For convenience, it is better to calculate the total run-off in second-feet. The mass curve at any specified month will be the total run-off up to that month in second-foot months.

Total run-off can also be calculated in acre-foot, million or billion cubic feet units depending on the size of the scheme, but second-foot month unit is most generally used.

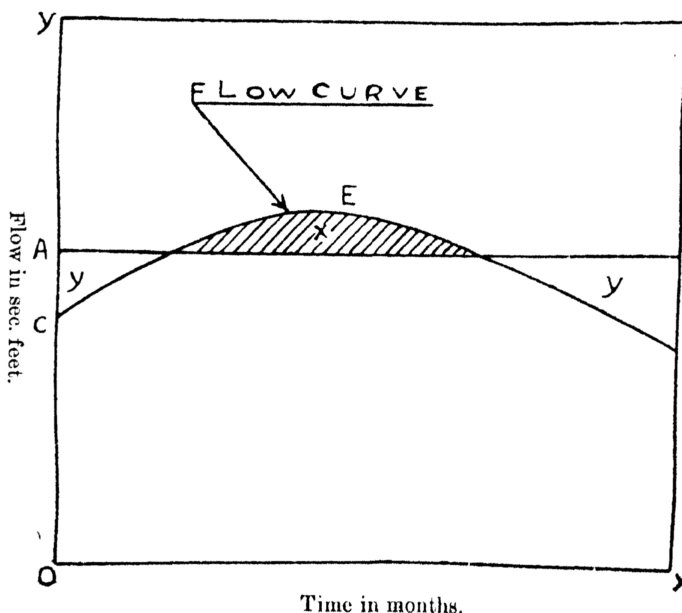
In the above figure, ADBC is the mass-curve. The demand for water in every month is the same. Hence, the demand curve such as OK, if it be plotted like the mass-curve, will be a straight line.

In order to determine the maximum demand which can be satisfied by utilizing all the water stored throughout the whole year, draw tangent to the minimum points at A, B and C. The slope of this line A C will give the maximum regulated flow which can be obtained for power purposes. For the maximum demand we see the reservoir is empty at A, B and C. If ordinates be drawn from the mass-curve upon the tangent AC, DE is the greatest among them, and hence DE is the maximum amount of water to be stored in the reservoir after satisfying the maximum demand.

Hence, the maximum length DE gives the necessary storage volume.

For any other demand curve OK, lengths of ordinates intercepted between the mass-curve and a line drawn parallel to the line OK, from points like A,B, or C, give the storage volumes at different intervals. If the slope of the mass-curve at any instant be greater than that of the demand curve, the reservoir is storing water ; if it be less the in-flow is less than the out-flow, and if it be equal, the inflow is equal to the out-flow.

Fig. 2.



The mass-curve should be drawn to take as long period as possible. The best result will be possible if the

period be more than 10 years. Monthly mass-curves are subject to errors because of their representing average rather than actual monthly flows. The errors generally occur at the beginning and at the end of the reservoir drawn down.

Another good process of calculating the maximum demand is to plot the rate of flow in second-feet along the ordinate on a time base.

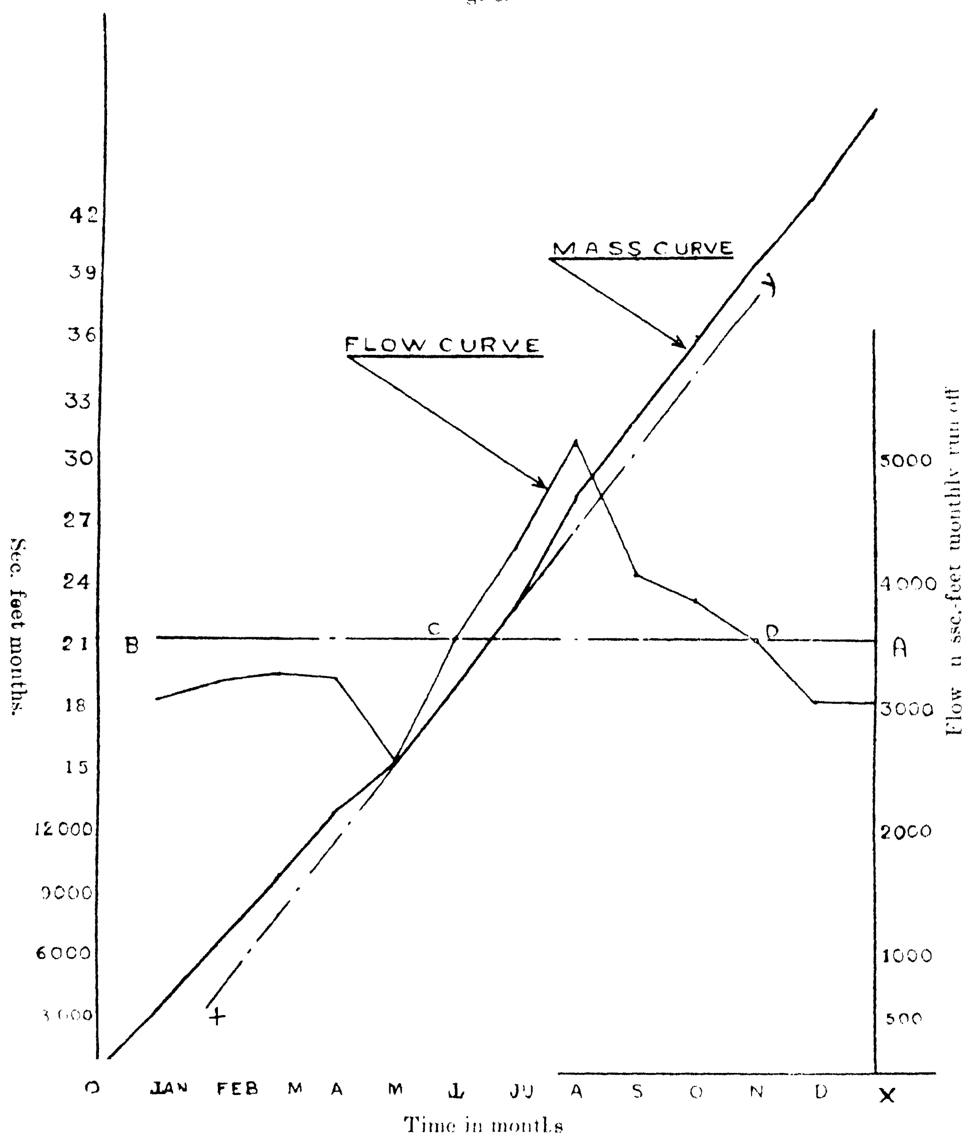
Draw the curve as in Fig. 2. Draw a straight line AB parallel to the base so that the portion of the area marked X is equal to the sum of portions marked Y and Y′. Then OA will re-represent to scale the maximum regulated flow which can be obtained for power purposes.

$$OA = \frac{\text{Area OCEDX}}{OX}.$$

Example:—For a typical river for the year 1917-1918 the flow is as follows:—

Month.	Second-feet. Mean Monthly Run-off.	Second-feet. Months Accumulative.	Month.	Second-feet. Mean Monthly Run-off.	Second-feet. Months Accumulative.
January ...	3000	3000	July ...	4200	22700
February ...	3150	6150	August ...	5100	27800
March ...	3200	9350	September ...	1000	31800
April ...	3150	12500	October ...	3800	35600
May ...	2500	15000	November ...	3500	39100
June ...	3500	18500	December ...	3000	42100

Fig. 3.



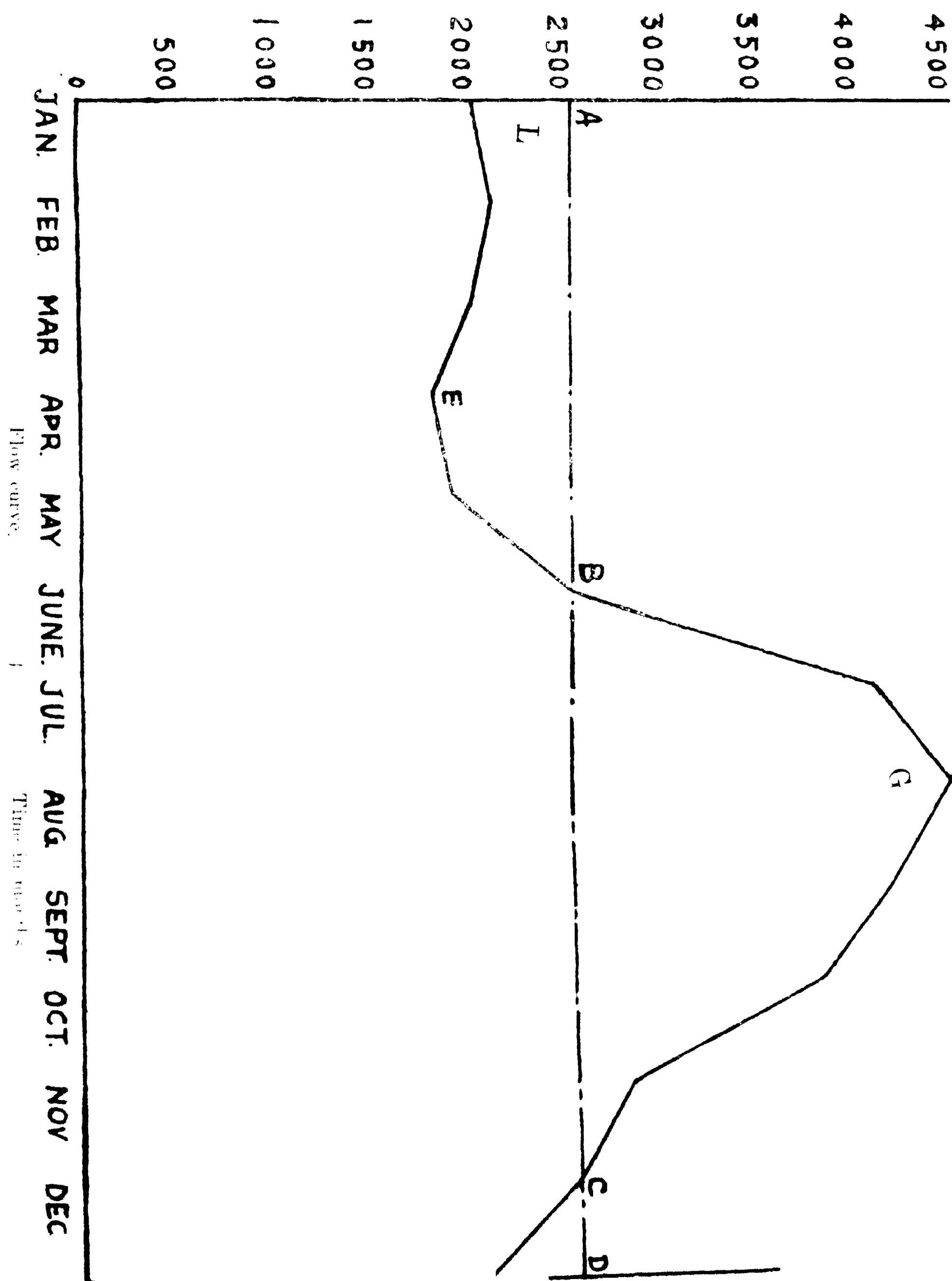
Plotting the points, both the sum-curve and the mass-curve are shown in Fig. 3. From the mass-curve the slope of the straight line XY gives the maximum regulated flow which is calculated to be 3750 second-feet.

From the flow-curve, AB is the line of average flow such that the area under AB above X axis is equal to the area under the flow-curve above the same line.

AX = 3500. Hence, from the flow-curve the maximum regulated flow is 3500 second-feet or cubic feet per second.

Example :—The run-off computation of a river after correcting for evaporation and seepage is as follows :—

Fig. 4.



Months.		Average monthly run-off.	Months.		Average monthly run-off.
January	...	2000 sec.-feet.	July	...	4100 sec.-feet.
February	...	2100 ,,	August	...	4500 ,,
March	...	2000 ,,	September	...	4200 ,,
April	...	1800 ,,	October	...	3800 ,,
May	...	1900 ,,	November	...	2800 ,,
June	...	2500 ,,	December	.	2500 ,,

Example:—

The average value of water to be consumed is 2500 cubic feet per second under a normal head of 40 feet. Calculate the reservoir capacity. Plotting the points we get the flow curve. From the flow-curve we find, the flow of the river is less than 2500 cu. ft./sec. from December to the beginning of June.

The reservoir is to accumulate water to meet the following demands :—

(1) To supply the water when the run-off is below the average value to be consumed.

(2) To supply the excess of water required when the head of the reservoir falls down.

(3) To store the water when the load is less than the daily average load.

Case 1 :—From the flow-curve we see the amount of water to be stored by the reservoir to meet the demand when the flow is less than 2500 cub. ft./sec.,

= area ABE + area CDF

= $5 \cdot 3 \times 500$ sec-feet months.

= 2650 sec-feet months.

Case 2 :—During the same period between December to the beginning of June, the head of water in the reservoir falls and it may be allowed to fall 25% below the normal. Hence, the average value of water to be consumed is more than 2500 cubic feet per second. Considering (40 + 90)/2 or 35 feet to be mean head let Q sec.-feet be the discharge required under the same head.

$$\therefore \; Q \times 35 = 2500 \times 40$$

$$\text{or } Q = \frac{2500 \times 40}{35} \text{ sec.-feet.}$$

Hence, water to be supplied by the reservoir per second more than the normal value,

$$= \left(\frac{2500 \times 40}{35} - 2500\right)$$

= 357·14 cu. ft./sec.

∴ Extra water to be stored = 357·14 × 6 or 2342·84 sec. ft. months.

∴ Total water to be stored by the reservoir—

= 2650 + 2242·84 ; or = 4892·84 sec.-feet months.

$$\therefore \; 2{\cdot}6 \times 4892{\cdot}84 \times 10^6 = \frac{A' + A''}{2} \times \frac{40}{4}$$

$$\frac{A_{,} + A''}{2} = 2{\cdot}6 \times 4892{\cdot}84 \times 10^5 \text{ sq. ft.}$$

$$= \frac{2{\cdot}6 \times 4892{\cdot}84 \times 10^5}{(1760 \times 3)^2} \text{ sq. miles.}$$

= 45 sq. miles.

Hence, the average area of the lake taken between its upper and lowest levels must be about 45 square miles.

From the flow-curve we have to see whether the collected run-off is sufficient to meet the demand of the cases Nos. 1 and 2. That is, whether the area of the curve above BC is sufficient to supply the demands Nos. 1 and 2. In the flow-curve area of the curve above BC—

= 500 × 12·5 or 6250 sec.-ft. months.

But storage necessary = 4893 sec.-ft. months.

Hence, the quantity of water which can be allowed to overflow every year—

= 6250 − 4893 or 1357 sec.-ft. months.

It would be impossible to consume water at the rate of 2500 sec. ft. from the collected run-off if the sum of the quantity of water demanded by the cases Nos. 1 and 2 be greater than the area above BC of the flow-curve.

This example illustrates the condition of a storage reservoir only large enough to equalize the yearly load. But there are some bad years when the collected run-off is comparatively less. To meet the demand for bad years the reservoir will have to store water. Hence, the reservoir area should be calculated from a flow-curve computed for 20 years.

The capacity of storage reservoir can be computed from the formula :—

$$C = \frac{500}{\sqrt[3]{R}}$$

Where, C = number of days' storage.

R = available rain in inches.

Messrs. W. J. E. Binni and H. Lapworth have lately suggested the following formula for calculating storage :—

$$S = \frac{15\,(\eta - d)^2}{r^{1.85}}$$

Where, S = inches of storage required to supply η inches per year.

d = the extreme minimum dry-weather flow reduced to inches per annum.

r = mean run-off also in inches per annum.

The storage required in gallons is equal to——

S × area of the gathering-ground in acres × 22,610.

Rate of draft from reservoir :—For any decrease in elevation of a reservoir level the rate of draft can be expressed :—

$$Q' = \frac{QH}{(H' - h)}$$

where, Q′ = draft in ft.³/ sec. ;

Q = average draft in ft.³/ sec. ;

H′ = normal head in feet, when full ;

H = average head in feet ;

h = fall in level below H in feet.

Mean velocity from Manning's formulæ for one condition of channel, in which R = 3·25 ft. and S = 0·00039, is—

$$v = \frac{1{\cdot}4858}{\eta}\sqrt[3]{R^2}\sqrt{S.}$$

$$\frac{1{\cdot}4858}{0{\cdot}025} = 59{\cdot}43\ ;\quad \sqrt[3]{(3{\cdot}25)^2} = 2{\cdot}194\ ;$$

$$\sqrt{0{\cdot}00039} = 0{\cdot}0198$$

$$= 59{\cdot}43 \times 2{\cdot}194 \times 0{\cdot}0198$$

$$= 2{\cdot}58 \text{ ft. per sec.}$$

Calculation of the area of a storage reservoir for a 24-hour regulation :—

(1) In calculating the area required for a reservoir we assume that the maximum variation of head will not be more than 25%. Hence, the water available for power development—

$$= \frac{A + A^1}{2} \frac{H}{4}$$

Where, A = area of the reservoir at normal head.

A^1 = area of the reservoir at 75% of normal head.

H = head in ft. under normal elevation.

Let us assume that for a particular plant the normal head is 40 ft. and that 2500 cubic feet per sec. is the volume of water required for power purposes at normal elevation. The head is lowered as the level in the reservoir falls.

If the lowest head $= 40 - \frac{40}{4}$ or 30 ft,

Average head $= \frac{40 + 30}{2} = 35$ ft.

And Q, the quantity of water required at this head—

Q is found thus—

$Q \times 35 = 2500 \times 40$

$Q = 2500 \times \frac{8}{7} = \frac{20000}{7} = \frac{2857}{}$ cu. ft.

Suppose it is found in the above case that the heavy load period is from 5-30 p. m. to 12 midnight and that the total load is 48460 kW. hr. and the average load $= 48460/6{\cdot}5 = 7452$ kW. as determined from the load curve.

Under the average head the power produced from 1 cusec of water $= 35/15{\cdot}7 = 2{\cdot}25$ kW. delivered by the generators if we assume that 15·7 cusec can generate 1 kW.

∴ Quantity of water required—

$$= \frac{7452}{2{\cdot}25} = 3310 \text{ cu. ft. per sec. at 35 ft. head.}$$

∴ Total water to be stored daily $= (3310 - 2857)\ 6{\cdot}5 \times 3600$ cu. ft.

The depth of water available for storage—

$= 40/4 = 10$ ft.

$$\therefore \frac{A + A^1}{2} \times 10 = (3310 - 2857)\ 6{\cdot}5 \times 3600$$

$$= 453 \times 6{\cdot}5 \times 3600 \text{ cu. ft.}$$

$$A + A^1 = 453 \times 6{\cdot}5 \times 720 \text{ sq. ft.}$$

$$= \frac{453 \times 6{\cdot}5 \times 720}{43560} \text{ Acres.}$$

$$= 49{\cdot}4 \text{ Acres.}$$

(2) **Calculation of Storage Reservoir** :—If there is no storage except one sufficient to equalize the 24-hrs. load, the average rate at which water can be drawn through turbines is equal to the minimum stream flow and this means that the slope of the line of draft will be equal to the minimum slope of the mass curve. With larger storage, the average rate of draft can sometimes exceed the rate of stream flow. If the normal head is 40 ft. and the allowable reduction in head is 25%, (*i.e.*), 10 ft., then the volume of this 10 ft. layer of water will be that which supplies in times of poor flow. Let this volume be represented by u. cu.ft. Take the point where the minimum flow ends and from there draw a vertical line representing u. cu.ft. and join this point with the point where the minimum flow begins. The slope of this straight line represents the minimum average flow that the stream together with the reservoir can supply. The point where this line again intersects the mass curve gives the date of refilling the reservoir.

Fig. 5.

Fig. (5) shows an enlarged portion of a mass curve where the flow is minimum. The periods of minimum flow is from Feb. 5 to the 7th March.

From the point where the minimum flow begins, *i.e.*, A, draw a line AB parallel to the continuous average draft line. Let C be the point where the minimum flow ends. From C draw a vertical line to meet the line AB in D. Then CD represents the quantity of water available for compensating the natural flow in time of low draft.

But DC on the vertical scale represents 530 million cubic ft. As we have already seen, the available reduction in head during this period can be 25% of the normal head or 10 ft.

∴ if A is the mean area of the reservoir during this period—

$$\text{Then } A \times 10 = 530 \times 10^6$$

$A = 530 \times 10^5$

or, 53 million cu. ft. = 120 acres.

As we see, AD produced meets the curve in B. Then B represents the date of refilling of the reservoir. At any point such as E, the ordinate between the mass curve and line of average flow required will give the average reduction in head to the scale on which CD represents 10 ft.

Hence, we see—

(1) the rate at which water may be using during the period of lowest water is 200,000 million cu. ft. for 67 days or 34·5 cu. ft. per sec.

(2) The draft of the reservoir began on January 1.

(3) The maximum depth of draft was on March 7.

(4) After March 27, the water flowing into the reservoir lake exceeded that used, and the reservoir began to be refilled.

(5) The reservoir was again full and the normal level of the lake was reached on March 13.

(6) The average reduction in head is given by dividing the Area ACD by AG and is equal to 5·9 ft., and, hence, the average head under which the turbines operated during the period of low water is 40 − 5·9 = 34·1 ft.

(3) **The capacity of storage reservoirs** can be computed from the formula :—

$$C = \frac{500}{\sqrt[3]{R}}$$

where, C = number of days' storage.

R = available rain in inches.

Messrs. W. J. E. Binni and H. Lapworth have lately suggested the following formula for calculating storage :—

$$S = \frac{15\,(\eta - d)^2}{r^{1 \cdot 85}}$$

where, S = inches of storage required to supply η inches per year.

d = the extreme minimum dry-weather flow reduced to inches per annum.

r = mean run-off also in inches per annum.

The storage required in gallons is equal to—

S × area of the gathering-ground in acres × 22,610,

(4) **Rate of Draft from Reservoir** :—For any decrease in elevation of a reservoir level the rate of draft can be expressed—

$$Q' = \frac{Q\,H}{(H' - h)}$$

where, Q' = rate of draft in ft.^{3}/sec.;

Q = average draft in ft.^{3}/sec., as found from mass curve;

H' = normal head in feet, when full;

H = average head in feet;

h = fall in level below H in feet.

(5) **Mean velocity** from Manning's formulæ for one condition of channel, in which $R = 3{\cdot}25$ ft. and $S = 0{\cdot}00039$, is—

$$V = \frac{1{\cdot}4858}{n} \sqrt[3]{R^2} \sqrt{S}.$$

$$\frac{1{\cdot}4858}{0{\cdot}025} = 59{\cdot}43;\ \sqrt[3]{(3{\cdot}25)^2} = 2{\cdot}194;$$

$$\sqrt{0{\cdot}00039} = 0{\cdot}0198$$

$$\therefore\ V = 59{\cdot}43 \times 2{\cdot}194 \times 0{\cdot}0198 = 2{\cdot}58 \text{ ft. per sec.}$$

(6) **Reservoir embankments** are generally of the earth and rock fill types consisting of rock or earth or both. They must be so designed that it will be able to resist the hydrostatic pressure of water which stands against them. No water should pass under or through them and that their exposed slopes are safeguarded against erosion due to rainfall. In order to prevent water from passing through or under a rock or earth-fill dam, an efficient cut-off is to be made below the foundation and a core-wall of some impermeable material in its body. The core is generally made of puddle clay or concrete wall. The core-wall is joined to the abutment and a drain-pipe is placed at its base on the upstream side passing through the abutment and discharging below the spill-way.

In order to safeguard the exposed surface due to erosion from rain, the downstream slope should be covered with grass turf having long roots and thick growth. In many cases the dam subsides due to heavy rain-fall and it can be remedied by true planting which offers the best security against subsidences due to heavy rainfalls.

When the embankment bed is not well-prepared and not thoroughly cleared of stump and brush, settlement will take place and may cause slide. It is good practice to keep suitable filling material in conveniently placed supply banks at various points along the embankment for ready use in case of needed repairs.

The feeder inlet into the reservoir must be kept free of sedimentation to the greatest obtainable depth ; snags and log jams should be promptly cleaned away.

(7) **Reservoir dam** :—The reservoir dam is generally an earth or rock-fill structure having masonry spill-way and sluice. It must be constructed with great care so that serious defects may not be developed. Precautions taken for reservoir embankments will also be applied for reservoir dam. Much care is to be taken for connecting the masonry portion with it. Undue leakage from a reservoir is the loss of useful power source. Any leakage found must promptly be prevented ; otherwise it will be of serious type afterwards. Core or wing wall prevents leakage. When the leakage is below the footings sheet piling along the upstream side may be restored to with good effect.

(8) **Care and Inspection** :—The embankment should be frequently inspected at least monthly, specially, following a heavy rain-fall. The feeder inlet to the reservoir must be kept free of sedimentation to the greatest obtainable depth. Snags and log joints should be cleared away, vegetable should not be permitted to grow rank along the flowage level.

(9) **The selection of material for a reservoir embankment** :—In this one side is constantly submerged and leaks are not traceable and are more difficult of access.

The desideratum is an impervious homogeneous mass called puddle consisting of such proportion of clay, small gravel and coarse sand that the voids in the mass are practically filled. Spread, gravel, sand and loam in 6 to 8 inches deep layers, well-watered and compacted with heavy iron rollers. *Clay* should be used but sparingly ; clay expands and contracts in the ratio of its degree of dampness to such an extent that no bank largely formed of it, no matter of what dimensions, represents stability or permanency.

Sand, confined in place, forms an excellent bank material, and when combined with gravel and loam, it yields the best.

An earth embankment with a concrete core consists of two parts,—the upstream and the downstream,—the exposure and wear of which differ greatly ; the former is submerged and subjected to wave action and whatever effect floatage and ice may have upon it, while the latter merely adds to the weight of the whole and is exposed to the rainfall. The two parts should be differently composed ; for instance, the upstream section might be built of the before-described puddle, and the downstream of a loose rock-fill with sand and loam washed into the voids and a

sufficient thickness of loam slope covering on the top of it to give sustenance to a grass turf. If the core-wall is of sufficient stability, such an embankment will be fully as effective as if it were formed entirely of puddle material, while, with rock available in the vicinity, its cost will be considerably less.

Defects in Reservoir :—

Troubles which are likely to develop are principally due to two causes : (1) settlement, (2) subsidences ; and the first is followed by the second.

Frequently the omission of core-walls in low embankments develop one or the other of the faults.

Remedy :—Add the core-wall or the substratum of an impervious wall on the water side of the embankment.

Cause.—The embankment bed is not well prepared, not thoroughly cleared of stump and brush.

Result :—Settlement will take place and may cause slides.

Cause.—Heavy rainfall causing subsidence.

Remedy.—Plant trees along side of the embankment.

Trouble.—Slides.

Cause.—These are traceable to an excessive ratio of clay in the fill.

Remedy.—Keep suitable filling material in conveniently placed supply banks at various points along embankments for ready use in case of needed repair.

Cause :—When the embankment has no core and the sluice walls no wings passing into the embankment, nothing will prevent leakage to pass through and along the sluice walls.

Remedy.—Add a core or wing wall.

Trouble.—Leakage is below footings.

Remedy.—Have recourse to shut down along the upstream side.

Tracing leakage.—To identify and trace the source of underground water such as leakage from reservoirs, canals, pipes, etc.

Fluorescein.—Mix fluorescein with supposed source of the leakage. One grain is sufficient to colour 100 tons of water.

Indigo.—One grain of indigo dissolved in sulphuric acid will colour a ton of water.

The colouration is more easily detected by placing the water in a very long test-tube and looking through the latter end-ways.

Pondage

When the reservoir is at some distance from the development, it is impossible to regulate the out-flow with sufficient accuracy to provide for sudden changes in load demand or to compensate for the varying run-off from the intermediate area. Therefore, utilize a regulating body of water upstream of the power dam, as pondage. It is needed directly at the plant. It refers to the storage for taking care of the daily fluctuation in the load curve. If there were no pondage, canals, flumes and pipe lines would have to carry the peak flow of water instead of the average. This may be drawn upon quickly to suit sudden changes in load demand and to compensate for inaccuracies in operation of the storage reservoir. When storage reservoirs are not provided, pondage is necessary to regulate the natural flow to suit the variation in daily or weekly load demand.

The hourly demand is quite variable, and the average demand during a work day is materially different from that of Saturdays and Sundays. The average weekly demand, however, is usually fairly constant. The duty required of pondage without storage is, therefore, usually that of regulating the weekly flow to suit the variation in load demand from the average weekly demand.

The rate of flow, the amount of water required from the forebay or from pondage, the maximum draw-dam, the net available head and the total cost, etc., are matters which usually settle *the size of and the best operating conditions for pondage* whether for high, medium or low dovelopments. The amount of pondage required in any given case will depend upon the load factor, or the amount of pondage may limit the size of the plant for any predetermined load factor depending, of course, on the type of development.

In the determination of forebay or pondage capacity just calculated one of the three conditions is assumed :—

1. A condition when the average daily rate of flow equals the daily rate of flow which is completely equalized by pondage.

2. A condition when the average daily rate of flow is less than the daily rate of flow which is completely equalized by the pondage

3. A condition when the average daily rate of flow is greater than the daily rate of flow which is completely equalized by pondage.

Troubles in pondage :—The defects which may diminish the utility of a pond are caused by sedimentation and ice.

Remedy :—Remedy for both lies at the dam whatever withdrawal provisions have been made and what has been said of the reservoir maintenance applies also in the case of pond. The principle is the same in both cases.

· Pondage supply is available only for a plant which does not operate continuously such as a mill working for sometime in the day. For a plant, producing electric current to some mixed day and night power and lighting load, the pond does not represent a supply source of any stability, but rather of weakness as in the event of its not refilling promptly, the effective head remains reduced until the natural supply from the drainage area reinstates the normal conditions. Only when the head is high and pondage area goes into thousands of acres does the pond partake of the utility of a supply reservoir.

Storage

Cauvery Power Scheme

This installation is one of the pioneer water-power plants in India dating as far back as 1897 started in 1902 with a capacity of 4000 H. P. ; this scheme mainly depended for its water supply on the river discharge. Gauging operations were, of course, carried out when the scheme was inaugurated, and it was found that the minimum discharge for any day was found to be 95 cusecs, whereas the quantity required was 181 cusecs, the deficiency to be made good, therefore, being 86 cusecs or 7430,000 cubic feet per day. The Madhavmantri dam forms a large reservoir, the capacity of which if the weir was raised $1\frac{1}{2}$ ft. was 313 million cubic feet. Consequently, it contained $\frac{313}{7.43}$ or 42 days' supply and the gauging operations showed that on only 32 days the water supply had to be supplemented by the reservoir. It, therefore, seemed certain that if the dry weather irrigation under the Madhavmantri dam was stopped and the weir raised $1\frac{1}{2}$ ft., enough water could be passed down to the falls to make good the deficiency. Further study of the rainfall data showed that in years except that like 1899, ample water supply was available for the generation of 4000 H. P. at the mines and, even in years of such low rainfall, the deficiency could be made good by utilizing the reservoir above the Madhavmantri dam.

Inaugurated thus, the Cauvery Power Scheme increased year by year in its capacity. From a mere Kolar load of 4000 H. P.

at start, the capacity of the generating station increased in 1907-8 to 10,000 H. P. comprising not only the increased load of Kolar Gold fields but also the load of Bangalore and Mysore, which were both electrified.

The additional power capacity provided in the power house could not be developed during the period of low water in the river. To compensate for this all dry weather irrigation had to be put an end to and use the water stored at the several dams on the Cauvery and its tributaries during hot weather.

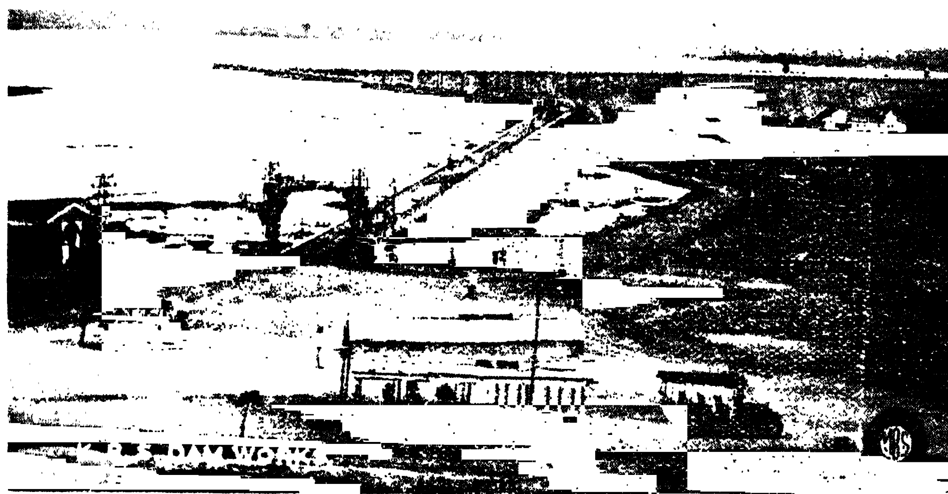
This was the commencement of storing water to meet the power demands and marks the beginning of a new era in the history of the Cauvery Power Scheme. The manner in which the power load was growing year by year showed that these temporary measures of converting water for power purposes would not be adequate to secure the reliability necessary to place electric power in the state on a sure and never-failing basis. *Provision for storage on a permanent basis and on a sufficient scale had to be made to enable the growing expansion of the generating plant at Sivasamudram.*

The Cauvery Reservoir Project was sanctioned in 1911. The dam construction was begun at Krishna Raja Sagara, 65 miles above the Cauvery falls, with the aim of providing a constant supply of 750 cusecs to Sivasamudram for the purpose of the increased power generation. *This constitutes what is known as the first stage of the Krishna Raja Sagara Dam.*

When the first stage was finished, 11,000 million cu. ft. could be stored in the dam, out of which 7,000 million cu. ft. could ensure a constant flow of 750 cusecs for the generation of 27,000 H. P. at Sivasamudram, which was the maximum demand at that time.

In spite of this, as the demand for power has considerably increased, reaching the respectable figure of 50,000 H. P., and as the irrigation problem had also to be tackled, the capacity of the dam was increased to 47,000 million cu. ft., and this ensured a sage supply of 900 cusecs constituting the 2nd stage of the Krishna Raja Sagara Dam, and a general view is shown in Fig. No. 6.

Fig 6.



Panoramic View of Krishna Raja Sagara Dam.

On the Cauvery river and its tributaries below the reservoir there are several dams with sluices which admit of very close regulation of the flow at Sivasamudram. The more important of these have been made as gauging stations and connected up by telephone, so that any variation in the flow can quickly be compensated for by changing the discharge at the reservoir. *This is particularly important as there is no storage either at the head works or forebay.* To provide against the contingency of a sudden drop in the river flow and to take care of extra load that may come upon, the discharge in the channels is maintained at 5% more than the previous maximum requirements. With projects of this class where the main storage is at a considerable distance above the head gates, it is necessary to provide at the penstock intake a definite amount of storage, based on the actual distance between the gates and reservoir in order to obtain the highest economy in the consumption of water.

Krishna Raja Sagara Dam

The chief objects of the scheme are :—

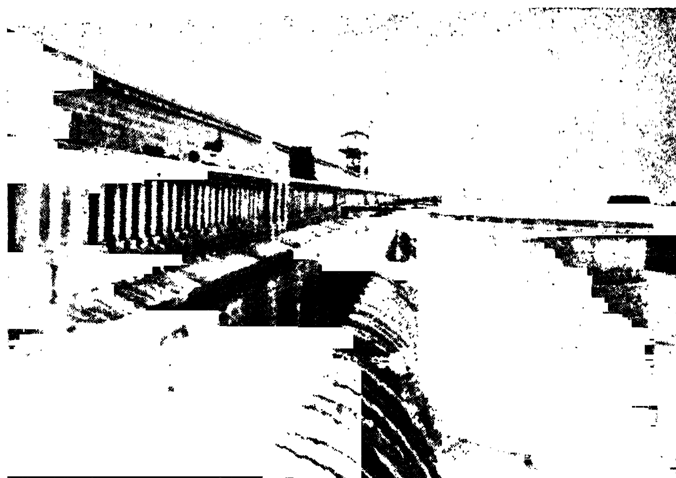
1. To ensure a steady supply of water for the generation of power at Sivasamudram.

2. To irrigate about 120,000 acres of land round Mandya, Maddur and Malavalli, which are subjected to chronic drought.

By the construction of the dam, the quantity of the power generated at Sivasamudram has increased from 10,000 H. P. to 50,000 H. P., and power supply has been guaranteed to the Kolar Gold Mining Companies and other large industrial concerns and for lighting in Bangalore and Mysore and also for the electrification of many towns and villages in the Mysore State.

Salient features of the dam :—

Fig. 7.



Waste weirs—Krishna Raja Sagara Dam.

1. Situation:— Below the confluence of the three rivers—the Cauvery, the Hemavathe and the Lakshmana Thirtha.
2. Catchment area :—4100 sq. miles.
3. Width of the river at the dam site 910 ft.
4. Greatest height of dam above foundation—140 ft.
5. Length of the dam including weir portion—86,000 ft., the weirs being shown in Fig. 7.
6. Road width at top of dam—14½ ft.
7. Bottom width at bottom of foundation—111 ft.
8. Maximum depth of storage—124 ft.
9. Maximum water-spread area—499 sq. miles.
10. Maximum length at the maximum water-spread—25 miles.
11. Capacity above bed of the river—48,335 million cu. ft.
12. Capacity above the irrigation sluices at +60 ft.—43,984 million cu. ft.
13. Quantity of masonry in dam—30 million cu. ft.
14. Cost per 100 cu. ft. of masonry—Rs. 31.

15. Quantity of earth work done in foundation—8·75 million cu. ft.

16. Cost per 1,000 cu. ft. of earth work—Rs. 55.

Sluices in the Dam

(a) *Scouring sluices* :—

(1) At 12 ft. above bed—8 of 6′ × 13′.

(2) At 50 ft. above bed—3 of 6′ × 15′.

(b) *Turbine sluices* :—

(3) At 50 ft. above river bed for power generation—4 pipes of 6′ diameter.

(4) At 40 ft. above bed—1 of 4′ × 8′.

(5) At 60 ft. above bed—1 of 6′ × 8′.

(6) At 60 ft. above bed—3 of 6′ × 12′.

(c) *For high level irrigation* :—

(7) At 80 ft. above bed—16 of 10′ × 20′.

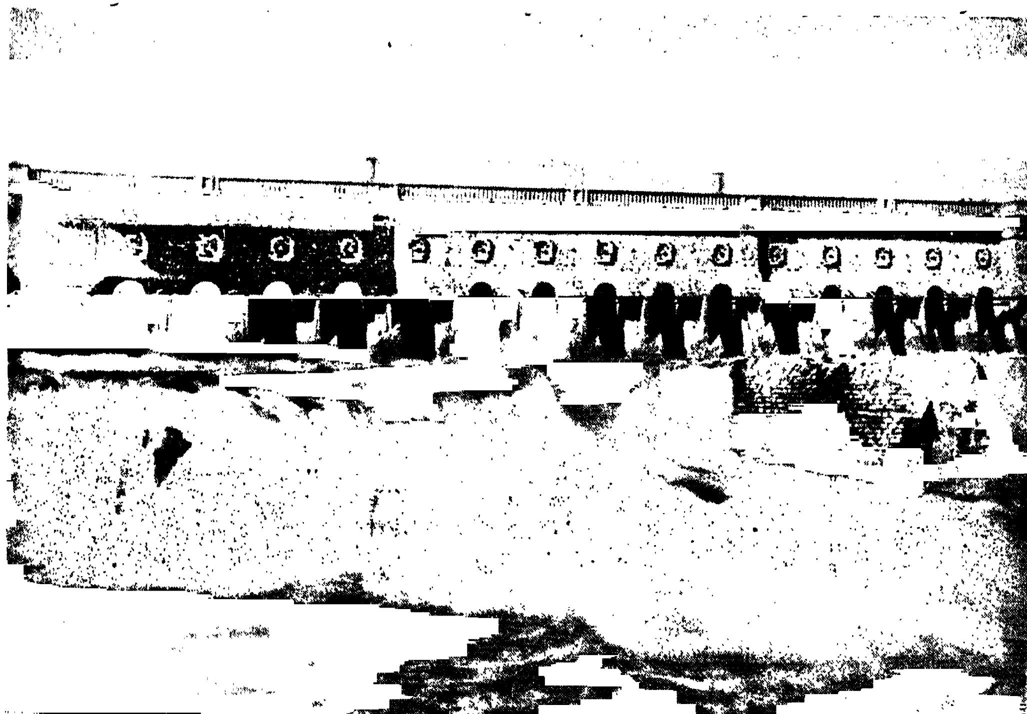
These are operated by electric power.

(8) At 103 ft. above bed—48 of 8′ × 12′.

(9) At 106 ft. above bed—40 of 8′ × 12′.

(10) At 114 ft. above bed—48 of 10′ × 10′.

Fig. 8.



Waste weir sluices—Shivasamudram Dam

These are operated automatically by the buoyancy of water.

Hydro-Electric Works

1. Distance of Dam to Siva—65 miles.
2. Height of effective waterfall for power generation—415 feet.
3. H. P. developed prior to construction of dam—10,000.
4. Increase in out-put with the help of storage in dam—40,000 H. P.

Other Important Facts

1. The reservoir scheme began to yield revenue from power works from the 4th year of its construction.
2. Nett revenue is 7·67%.
3. Beside power purposes, the increase of produce is expected to be as under—

Yield or gross produce after irrigation is developed.

(1) 40,000 acres of sugarcane	...	Rs. 200 lakhs.
(2) 40,000 acres of paddy	...	,, 32 lakhs.
(3) 40,000 acres of miscellaneous	...	,, 20 lakhs.
Total 120,000 acres	...	Rs. 252 lakhs.

Table showing important features of various reservoirs in and out of Mysore State.

Particulars.	K. R. S. Dam.	Andhra valley, Bombay.	Bundardhara, Bombay	Vanivilasa Sagar, Mysore.	Dharma Dam, Bombay.	Lloyd Dam, Bombay	Nizam Sagar, Hyderabad (Deccan).
1. Catchment area (sq. miles.)	4.1000	..	49	2,075	156	128	8,876
2. Maximum rain fall per annum in H.	250	..	200	150	144·9	250	...
3. Maximum discharge in cusecs.	2,90,000	...	...	...	60,400	...	525,000
4. Length of dam in ft.	8,600	1,800	2,300	1,330	4.480	5,333	17,490
5. Height of dam in ft.	130	190	270	142	92	190	115·0
6. Width of basin in ft	111	148	234	150	60·3	124	...
7a. Gross capacity in million cu. ft.	47,335	9,000	12,891	30,000	8876	24,198	35066
7b. Effective capacity ...	43,934	...	......	...	...	..	25,446
8. Cost in lakhs of rupees.	256	38	73	30	27·3	172	365·7
9. Cost per million cu.ft. stored in rupees	564	422	565	100	310	71·8	1,278
10. Masonry in million cu. ft.	30	14	12·65	10	...	21·5	30·07
11. Cost per 100 cu. ft. 1 masonry.	31	...	...	...	...	...	...
12. Capital cost incurred during 15 years of start.	13,619,197	...	97,56,000	..	81,27,000	27,39,400	...
13. Revenue earning per year	2,240,945	...	49,000	...	191,000	114,17,000	...
14. Per cent. return ...	5·26	...	·5	...	2·35	*Nil*	10

Storage—Pykara

It was explained in the beginning of this section that a watershed is an integral part of a hydro-electric station even if no storage or carry-over is proposed, *i.e.*, even when it is developed only up to the minimum year for minimum seasonal waters may not occur in the minimum year. Also, when the project forms the only source of supply to a transmission system, as is the case with Pykara at least for the first few years, it is generally uneconomical to dovelop it above the minimum year. It requires storage to be carried from year to year and this is not always economical since the water may only be required once in a period extending over several years, *i.e.*, at rare periods of extreme drought. The big reservoirs necessary for this storage would be costly and would necessitate fixed charges to be met annually on unproductive work. But, however, on a second source of power being added to the system, either a steam or hydro-electric plant situated in an entirely different watershed, the original plant can be safely developed to a point in excess of minimum conditions if economically feasible. The diversity between the various run-offs may often cover—within, of course, certain limits—any individual deficiencies. The position of Pykara in a possibly larger and more extensive power system of the future, probably necessitates due consideration to this point even in the choice of initial designs. As long, however, as Pykara is the only generating plant on the system, operations will be strictly on a basis of minimum run-off for each catchment. In other words, reservoirs designed for a minimum year condition will store and distribute the run-off for that year without spilling, but those designed for less will spill every year. Similarly, those designed for a 90% year will store and distribute the run-off for that year without spilling, but will not fill when the run-off is less than that for the 90% year.

To take full economical advantages of the Pykara Scheme storage should be gradually developed to suit the varying load conditions. It may also be pointed out that this catchment is admirably suited for such a development of the capacity following closely the load demand curve, whereas such methods are not possible in many projects.

In general, owing to the nature of the ground in the Pykara catchment, high dams could only be constructed at a very high cost, whereas conditions were found to be more favourable for lower structures and the required amount of water could still be stored. When it is realised that given equal conditions two 50 ft. masonry dams of the gravity type can be

constructed for a much lower cost than one of 100 ft., the economic advantage is obvious. In fact, even a reasonable amount of economy could be sacrificed and the design of a series of smaller storage reservoirs preferred if the bigger ones were not as costly as they really are. The economic advantages of the flexibility of the scheme involving storage are, therefore, obvious and this one feature adds to the value of the scheme.

The various dam sites are (*i*) Mukruti, (*ii*) Pykara basin, (*iii*) Porthmund, (*iv*) Sandy Nalla, (*v*) Person's Valley, (*vi*) Glen Morgan, and the exact topographs of these sites are seen from the plan. An account of the estimates and technical details of these future watersheds may not, perhaps, deserve a place here, but, however, an attempt will be made here to understand and account for the choice and order of development from an economic point of view.

The following table gives a rough idea of the unit cost of development and capacity of each site :—

Reservoir.	Run-off catchment per minimum year in millions of cu. ft.	Approximate ratio of unit cost.
Mukruti ...	1080	15
Pykara ...	1447	27
Porthmund ...	459	35
Sandy Nalla ...	627	43
Parson's Valley ...	396	50
Glen Morgan ...	66	81

It will be seen that where operating conditions permit, the most economical policy to follow is to develop *the cheeper storage sites for run-offs greater* than that of the minimum year and the more *costly* site for *the minimum run-off or less.*

It will be seen that Mukruti is easily the most attractive site, and it is but reasonable to expect it to be taken up at first. But, the order of the choice must *satisfy practical operating conditions,* besides ensuring the necessary continuous river flow and involving the least annual charges.

The development of Glen Morgan dam and the forebay had to be taken up initially, giving only a second place to Mukruti, *solely on practical operating conditions.*

A limited storage was necessary to provide water supply for construction purposes; also a small amount of storage was found safe to ensure a continuous flow to the auxiliary plant now in operation for the dry months. *Low initial investment* rather than *low cost per unit* governs the decision in this case. Glen Morgan was the site chosen because it will be also needed later on as an alternative water supply for cooling the transformers and supplying the camp with water. Also it was mentioned before that it could be used as a basin after the completion of Sandy Nalla. No other site could fulfil these conditions so well. Owing to the high—the highest of all—unit cost of this site, the *capacity is much below the minimum run-off* of the catchment. The capacity is about 25 millions cu. ft. effective and the height of the dam is 45 ft.

Similarly, a regulating forebay is necessary and it has been designed for a capacity of nearly 55 million cu. ft., so that the forebay and the Glen Morgan dam could take up the initial load for the first stage and, in fact, the first few years. No accident to the Pykara flume is anticipated, of course, but should the unforeseen happen, the capacity of the forebay should provide plenty of time for repairs.

Mukruti and Porthmund were chosen next in the order named, as it was found that this combination gave the lowest annual charges over a period of years in spite of the fact that there will be a certain amount of idle capacity at Mukruti for some time. The construction of these will ensure a regulated flow of 105 cusecs.

Sandy Nalla is placed fourth, more for *reasons of expediency than economy*. It is considered that at this stage of the development an alternative feed to the forebay is justified.

Moyar might probably be considered next. This installation might be considered as a cheap storage site since, by generating power from the tail race of the Pykara plant only a smaller draw-off from the existing reservoir will be needed for a given amount of power.

Pykara is a very good reservoir site, but the unit cost would go very high for points above 100 ft. But to ensure the contemplated flow of 150 cusecs only 900 millions out of the ultimate 1550 million cu. ft. of its capacity need be developed.

Prason may probably be abandoned due to its high unit cost.

The above order with corresponding figures justifying their respective places are tabulated below and may be of some interest :—

Reservior.	Run-off of catch per min. year m. cu. ft.	Eff. Res. Capacity.	Unit cost approx. ratio.	Flow in cusecs.	E. Q. K. W.
Glen Morgan ...	66	25	81	33·5	6700
Forebay ..	...	55	...		
Mukruti ...	1080	1200	15	92	18400
Porthmund ...	459	400	35	105	21000
Sandy Nalla ...	627	450	43	128	25600
Moyar ...	...	...	...	128	38200
Pykara ...	1447	1550	27	165	44300

CHAPTER VIII

ELEMENTS OF HYDRO-ELECTRIC PLANTS

Hydro-electric engineers should be thoroughly conversant with the elements of such plants. They must visit a number of places and scrutinize the elements of similar plants in other parts of the country. To facilitate their study they should note the details stated as follows and have a quantitative idea of the details, if possible.

1. *Headworks or head race* :—Constructed at the point where nature ends and man begins, for diverting water to the canal or pipe line. It may be a main impounding dam or a dam to raise the height of the water to create an artificial head.

Fig. 1.



General view of the headworks, Rampur Kashmir.

Fig. 2.

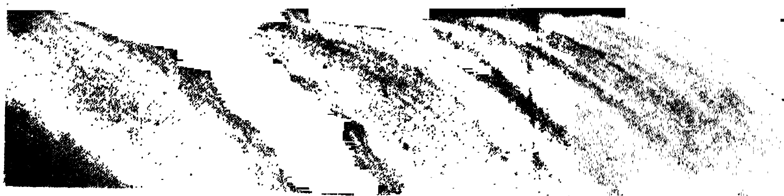


A view of the headworks from the river, Rampur, Kashmir.

2. *Conduits* :—

(i) Open conduit :—Canals, flumes, their function is to keep the forebay supplied for the source of water and to lose as little of the fall as possible in their course.

Fig.



Water entering open channel at headworks, Rampur, Kashmir.

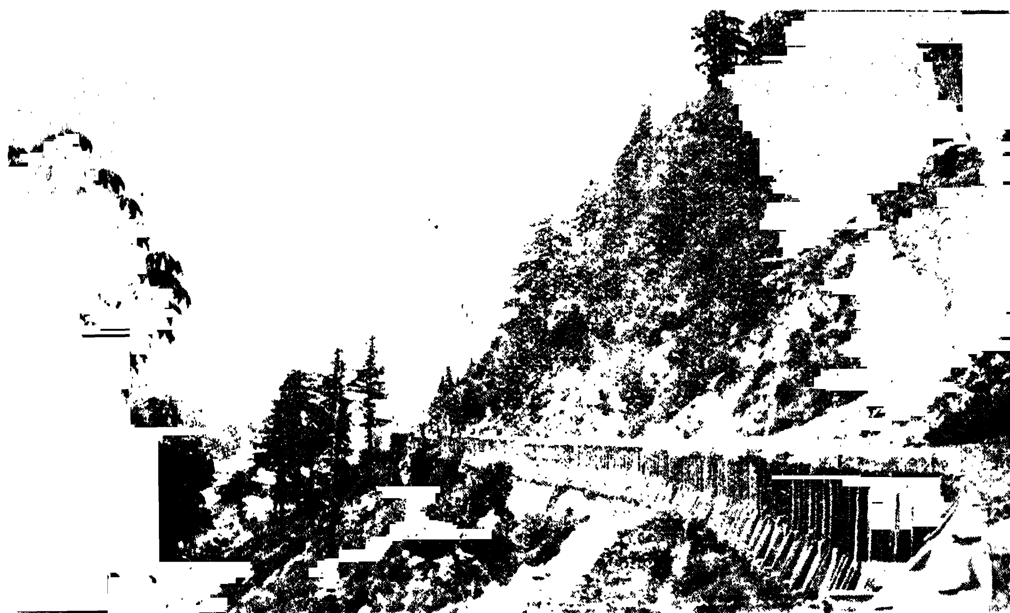
Area of intake	...	...	Square feet.
Area of conduit with full discharge	...	...	,, ,,
,, ,, ,, part ,,		...	,, ,,
Wetted perimeter	...	...	Feet.
Length of conduit	...	...	,,
Depth with full discharge		...	,,
Slope of conduit	...	...	Feet per 1,000 ft.

(*ii*) *Closed conduit* :—Tunnel through a spur shortens the channel appreciably, and saves a little head and may further serve to avoid bad ground.

Area of intake	...	...	Square feet.
Area of conduit	...	...	,, ,,
Length of conduit	...	...	Feet.

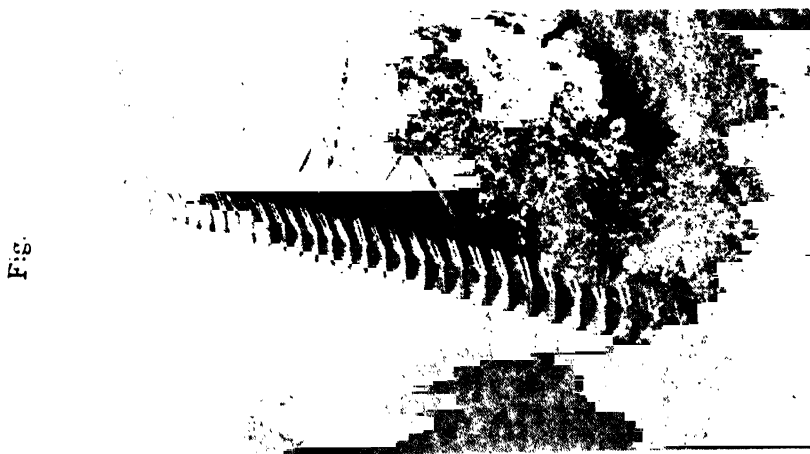
3. *Flumes* : -

Fig. 4.



Wooden flume channel, Mahora, Kashmir.

Fig.



Wooden flume channel, Mahora, Kashmir.

(1) *Open flume setting*:—

Net rack area per unit	...	...	Square feet.
Intake area	...	...	,, ,,
Dimensions of flume	...	...	,, ,,
Submergence of turbine	...	...	Feet.
Type of turbine	...	...	,,

(2) *Encased setting*:—

Type of casing (spiral, cylindrical, etc.)			
Area of inlet	...	...	Square feet.
Diameter of casing	...	...	Feet.
Length of casing	...	...	,,
Area of spiral at several points		...	Square feet.
Type of turbine	...	...	

4. *Forebay* is the reservoir which feeds the pipe system or penstock.

Area with full pond	...	...	Square feet.
Net rack area	...	...	,, ,,



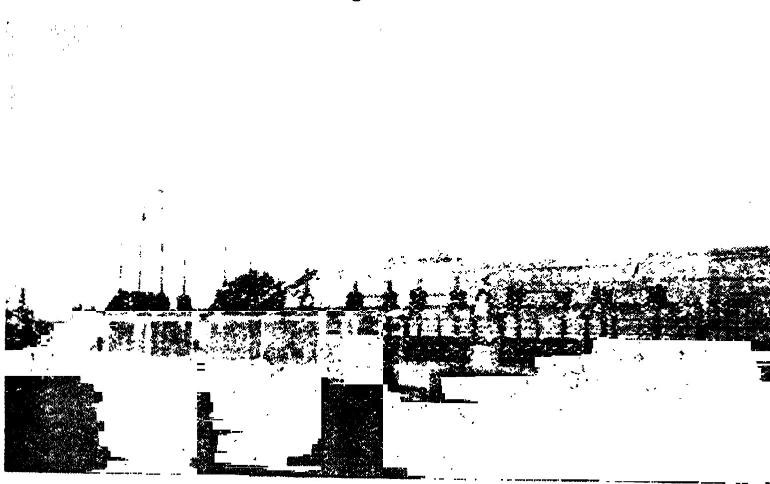
Fig. 6.

View of Ranbir Canal showing part of the Forebay.

Net area of intakes	...	...	Square feet.

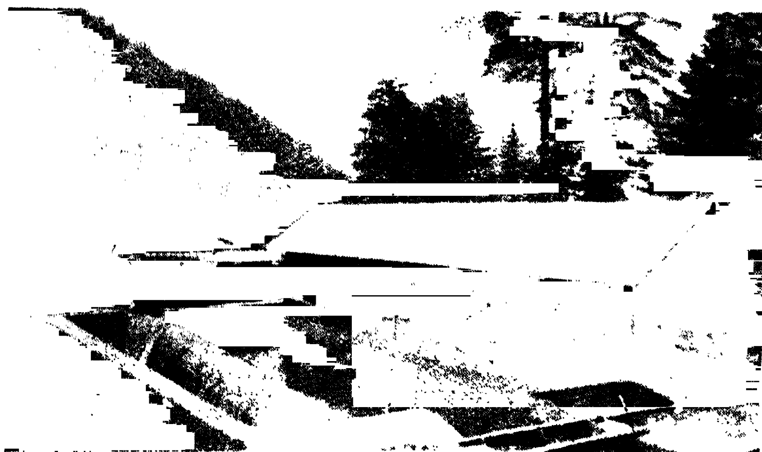
The forebay is generally the enlarged body of water just above the intake. It may be the pond formed by the diversion dam or it may be the enlarged section of a canal which is spread out to accommodate the required width of the intake. Generally a deflecting device is provided at an angle of 30° or 45° to the direction of flow to divert ice and trash away from the intake.

Fig. 7



Forebay and sluice gate, Jammu.

Fig. 8.

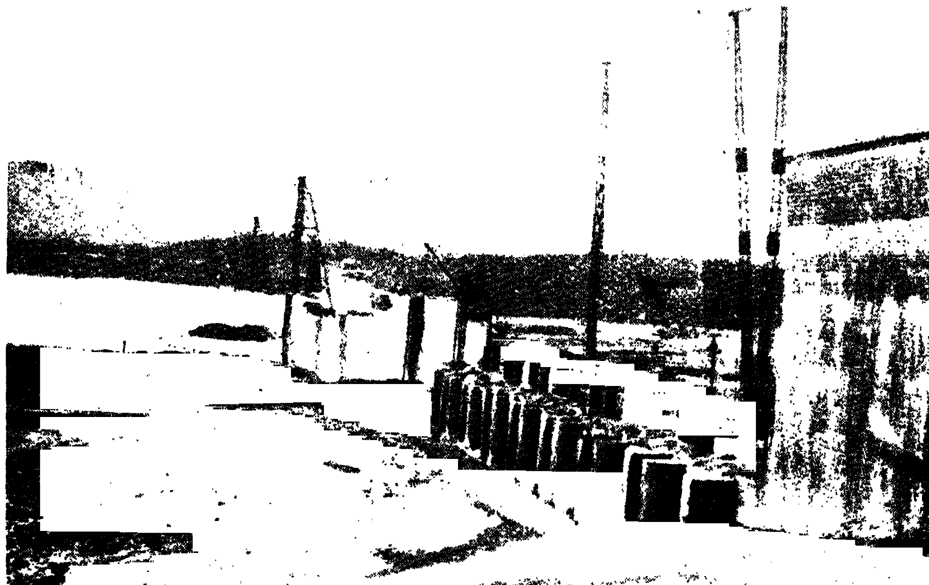


Forebay, Mahora.

Submergence of top of intake with full pond			Feet.
Elevation of surface with full pond			,,
Spill-way length	...	...	,,
Elevation of crest of spill-way		...	,,

5. *Reservoirs*:—Collecting, balancing or regulating reservoirs. These may be natural lakes or artificial ponds.

Fig. 9.



The dam and the lake in front Mittur.

Fig. 10.





Reservoir for the upper power-house, Darjeeling.

The reservoir dam is generally an earth or rock fill structure with masonry spill-way or sluice.

Name—

Distance from power plant in miles.

Area of surface of full reservoir in acres.

Area of each foot of depth in acres.

Draft in feet.

Capacity in cubic feet.

Outlet area in square feet.

6. Penstock or pipes to convey water to the wheel :—

Inlet area in square feet.

Net or/and average area of pipe line in square feet.

Length of pipe line in feet.

Area at turbine connection in square feet.

Each pipe line has been bifurcated into 2 parts just near the power-house, each supplying water to one turbine.

Fig. 11.



Penstock—Pykara.

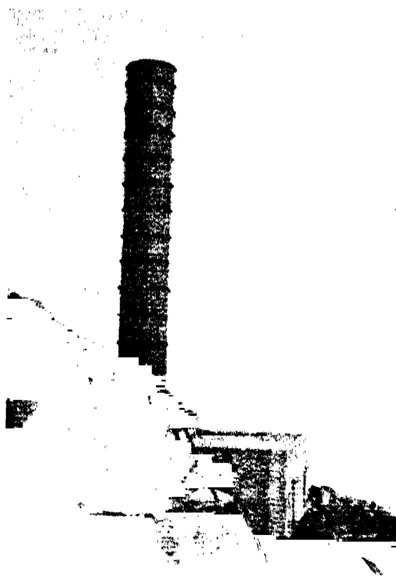
Fig. 12.



Two pipes divided into three for supplying 3 impulse turbines, Pykara.

7. *Siphon* :—In crossing minor streams, siphons of the inverted type are sometimes very serviceable. When the line is to be laid over ground which is higher than the hydraulic gradient, a siphon is commonly use

Fig. 13.



Surge tower, Pykara.

8. *Surge tank* :—A surge tower or stand pipe is generally connected to the pipe system at the junction to secure the safety of the weaker sections in the pipe line by giving a stand-by supply of water during the period occupied by the long column of water in accelerating upon an increase in demand, and to provide a relief chamber for the excess tending to injure the lower section when the demand is suddenly checked. It is located as close as possible to the power-house to reduce the length of penstock to a minimum, and preferably on high ground, to reduce the height of the tower.

Distance from turbine	...	Feet.
Type (open, tank, differential, etc.)	...	... ...
Net area at pipe-line connection	...	Square feet.
Net area at tank	...	,, ,,
Net area at throttle, if any ...	...	,, ,,
Elevation of connection to pipe line	...	Feet.
Elevation of top	...	,,
Elevation of overflow ...	...	,,
Overflow area with 1 ft. discharge depth	...	Square feet.

9. *Gates and valves* :—These, used to control the amount of water at the entrance, directly control the entry to the chamber. These are (1) parallel gate valves feeding each pipe, or (2) wiring gates or *butterfly valves* as are in the pipes. The intake must be guarded by a trash rack consisting of flat iron bars set in an iron frame and secured to the pipe intake.

Fig. 14.



Butterfly valve.

Fig. 15.



Air valve at Mahora.

Sometimes due to partial failure of pipe lines or otherwise water is removed from the pipe line faster than it can be supplied from the forebay. In such cases air must be admitted to the interior of the pipe to prevent the formation of a vacuum, with its consequent danger of the collapse of the pipe. Stand pipes for air inlets are better than air inlet valves, but are often too expensive and for this air valves are generally used.

Air valves :—Generally two kinds of air valves are in use, one is for use on pipe lines which followed the contour of hilly country, where any air may accumulate at high summits and obstruct the flow of water. At Nainital the pipe line has been laid without *crests of dips* and hence no such air can accumulate.

The other kind, known as poppet valve, is used to permit air to enter when water is being drawn off and thus eliminate any danger of collapse from vacuum forming in pipe lines, as for example when gates are closed. They are so provided at Nainital—to allow air to escape when pipes are being filled. The valve remains open until water reaches and lifts the copper float and closes the same, after which it remains closed while the pressure is on.

Fig. 16.



Valve house for controlling intake, Pykara.

Vents, admitting air to the sluice below the control, have reduced the vacuum to a considerable extent and should always be provided. It is not possible to instal enough vents at the control to destroy entirely the tendency to a vacuum.

10. *Silt traps or settling tanks:*—These are used to ensure clear water to the wheels. Proper allowance should always be made for estimated reduction in capacity from silting, either by providing excess capacity to retain the silt for a long period or by control of other storage sites for future use. Note that no effective means for removing enough of the silt to restore the usefulness of the reservoirs has yet been found (*vide* p. 226).

Fig. 17.



Left side of silt reservoir—Shiva Samudram.

Note that if the velocity is reduced to 0·5 feet per second practically all particles having a diameter greater than 0·07 millimeter will be deposited, and that particles remaining in suspension with this velocity will be too fine to cause appreciable wear on the turbine parts. In the design of the settling basin great care should be taken in distributing the velocity throughout the cross-section, so that all the parts of the cross-section have practically the same velocity.

Fig. 18.



Side view of silt reservoir, Shiva Samudram.

The sand which collects in the bottom of such a basin is drawn off through a number of mud valves fitted with horizontal discs.

Fig. 19.



Silt basin at Rampur, 800′ × 100′, Mahora.

11. *Turbines* :—Type (vertical, horizontal, impulse or reaction, open-flume or encased, spiral casing or cylindrical, needle or deflecting nozzle, swivel or cylinder gate, single or multiple runner, outside or inside gate mechanism, etc.)

Rated capacity under	...	ft.-head	... b. h. p.
R. p. m. at rated capacity	...		... r. p. m.
Maximum gate opening	...		... Inches.
Maximum net nozzle area	...		... Square ins.

Turbine runners :—

Reaction or impulse		
Kind of buckets		
Material of runner or disc.		
Inlet of depth of buckets	...	... Inches.
Total discharge area of buckets		... Square inches
Nominal diameter of runner		... Inches.
Maximum discharge diameter		... ,,

Fig. 20.



Turbine, governor and alternator during erection.

12. *Governors*:—Apparatus to convert mechanical energy of the turbine to—

Capacity	Feet-lbs.

Fig. 21.

Type (oil-pressure, water-pressure, etc.).

Governing system (unit or central).

Pressure system (open or closed).

Working pressure in lbs.

Closing time (full stroke) in seconds.

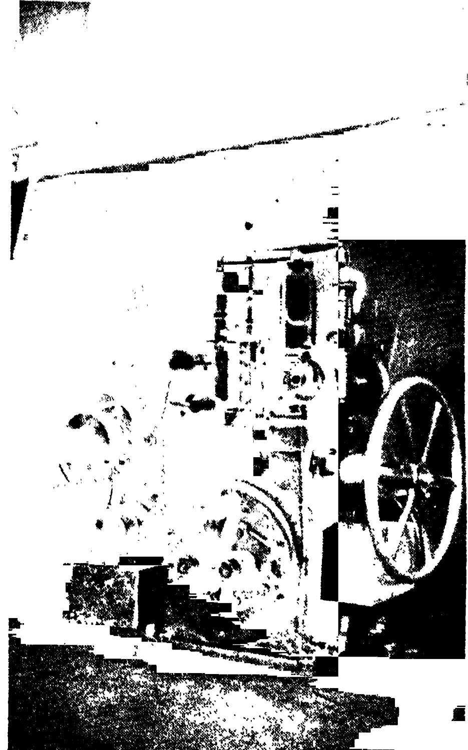
Opening time (full stroke) in seconds.

Difference in speed between no-load and full-load in per cent.

Capacity of oil pump in gals. per min.

Volume of pressure tank in cubic feet.

Volume of receiving tank in cubic feet.



Gilbert Gilkes Governor, Jammu.

13. *Draft tube*:—Where reaction wheel is used, the casing is entirely filled with water and is continued in the form of a draft tube which discharges beneath the tail race. It utilizes by vacuum or suction any further fall, called the draft head, below the wheel, so long as it is within the limits of the water barometer.

Area at runner discharge ...	Square ft.
Area at outlet ...	,,
Maximum gross draft-head ...	Feet.
Minimum gross draft-head ...	,,
Length of draft-tube ...	,,
Area of tube ...	Square feet.

14. *Tail race* :—The water from the wheels after doing the useful work goes to the tail race which carries it away either back to the stream from which it came or sometimes to a different one.

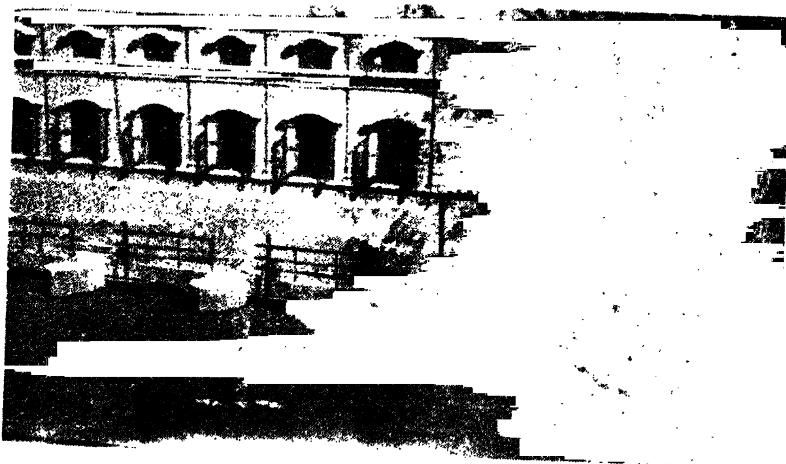
Fig. 22.



Tail race joining the spill-way, Mahora.

If the power station is on the bank of the stream, the tail race is formed in the substructure of the station (*vide* Fig. A, p. 90). The draft tubes must be water-sealed under all conditions and that free discharge of water is not impeded in any way. Generally, *stop logs* are provided at the point of exit of the power station to isolate the suction pits for inspection and repair. For facility of repair it is better to discharge all the units in the power station into a series of pits each having its own exit at the side of the station. A velocity of 3 to 4 feet may be allowed in the tail race.

Fig 23



Tail race, power-house, Jammu.

Cross-sectional area at draft-tube	...	Square ft.
Depth	...	Feet.
Wetted perimeter	...	,,
Surface slope	...	Ft. per 100 ft.
Diameters and lengths of piping	...	Inches or feet.
Fly-wheel effect or respective parts	...	lb. ft.
Synchronising device	...	
Load limiting device	...	

15. *Generators* :—Electrical energy at the voltage required for transmission.

16. *Switchboard containing* :—

(*a*) Electrical regulating devices.

(*b*) Electrical recording devices.

and, (*c*) Electrical operating devices.

Housing and supports for the hydraulic and electric apparatus.

17. *Transformers* :—Transmission system to transmit power to its point of ultimate use.

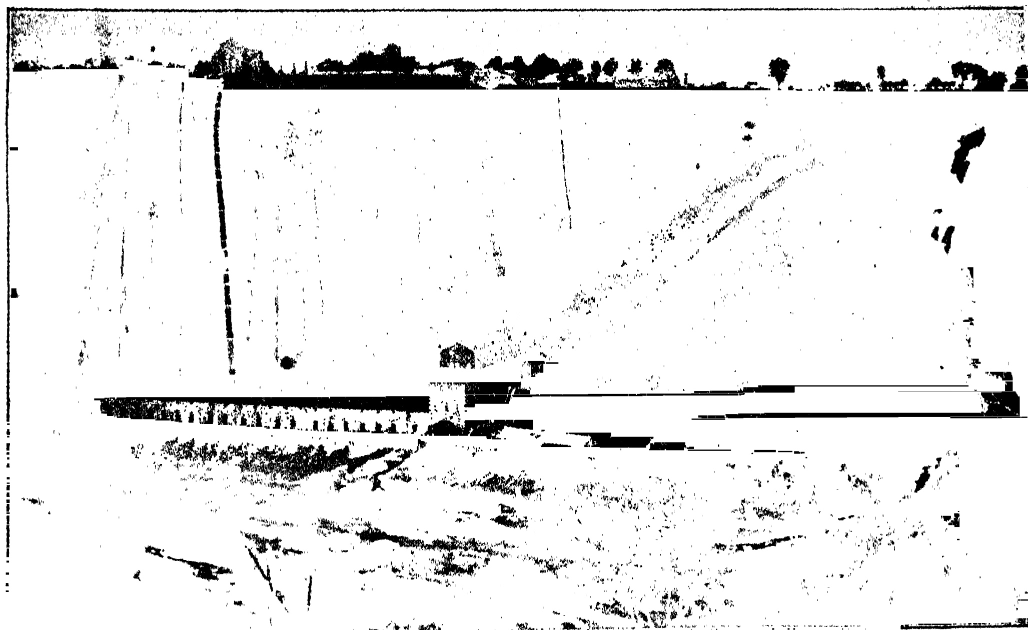
Protective apparatus :—

Control boards :—

Bus bars :—

18. *Power House* :—The construction and arrangement of

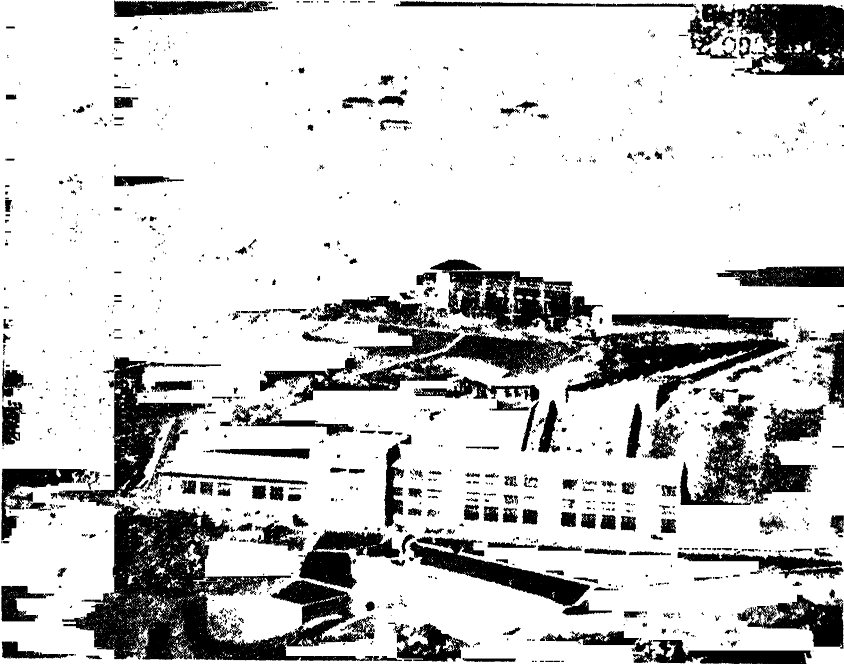
Fig. 24.



Power station at Shiva Samudram.

the power-house should be very carefully done. It must be

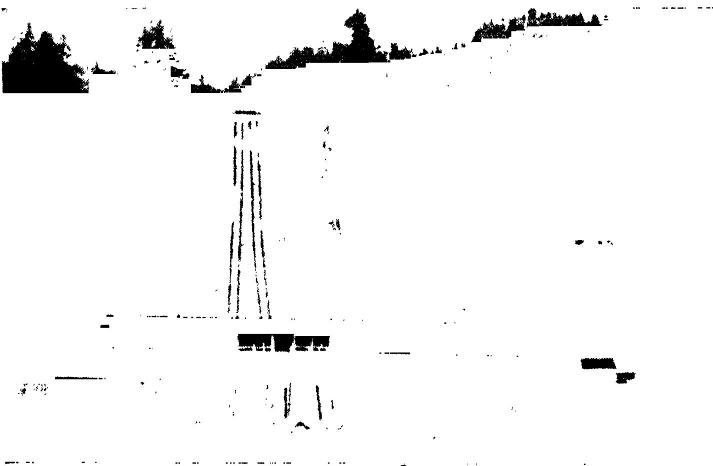
Fig. 25.



Shanon power-house and substation, Mundi.

(1) æsthetic, and (2) scientific. The position, foundation, the materials of structure, the arrangement of machinery and switch board should be minutely examined and laid down in the plan. Too much care cannot be given in the proper construction of the power-house.

Fig. 26



General view of the power-house, Mahora, Kashmir.

The General Requirements :—*The site for the intake* must be carefully chosen and the angle of its mouth with respect to the direction of the current must be so arranged as to avoid the accumulation of debris. Intakes to conduits are located at the upper end of the conduits or portion of the conduit.

. The best position of the intake works is such that water enters it at right angles to the natural flow of the stream. Piers extending out from the mouth of the intake form support for the trash racks which are placed there to keep large pieces of debris from the turbines. It is usual to have vertical stop log grooves cast in the concrete near the ends of piers. *Stop logs* are then cut to fit across the span between piers, so that they will easily slip into the grooves.

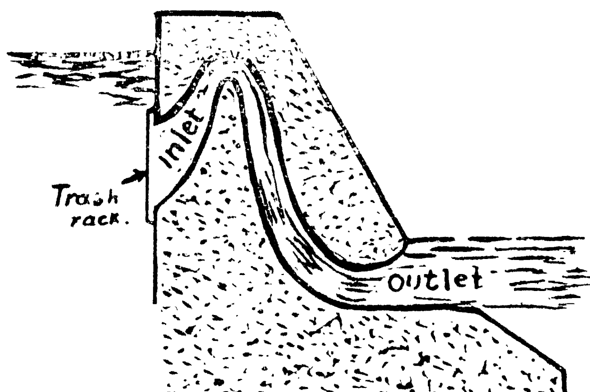
Fig. 27.



Stop logs at the headworks Mahora.

There must be (1) structural stability ; (2) limitations of velocity which must be confined within economic and practical limits ; (3) hydraulic efficiency—the shape of the water passage must be such that the transformation of static head to conduit velocity is gradual and entails the smallest practical eddy losses ; (4) practicability of operation—all apparatus should be reliable and reasonably quick of operation.

Fig. 28.



Mark the position of trash racks.

The intake gates and valves may be of the following types:--(1) Sliding Gates—these are limited in capacity by the effort that is required to move them against frictional drag set up by heavy water pressure; when these are equipped with roller guides, the capacity per gate is much increased; (2) Pivot Gates; (3) Roller Bearing Gates—the rolling gate is supported on a cylindrical drum which can be rolled up a stop inclined by various means; (4) Cylinder Gates; (5) Butterfly Valves; (6) Needle Valves—adopted for high pressure duty. Needle valves are preferred for extremely high heads. The type to be used depends upon the size of the opening, the head on the gate and the operating conditions.

Intake :—It is a basin of larger flow section than that of the diverting conduit proper at which it terminates. It may be confined between artificial embankment and wall. *Its function* is to assemble the water for the purpose of diversion, in a pool of considerable area. It thus affords facility to manipulate floatage which may enter in order to prevent its passage into the conduit. It further induces the flow velocity to a minimum. There may be several intakes to a system of conduits.

The intake includes all the works necessary for drawing the water from the storage basin or river and delivering it to the canal, pipe lines or other water conductors. If there is no pipe line or canal, the intake also forms the forebay. The design and location of intakes are governed very much by local conditions.

Intakes are of two types :—(1) Low pressure intake, and (2) high pressure intake. *With low head falls* the intake generally forms a part of the dam or power-house. The upstream bay forms the gate room, the gates and screens are thus installed indoors. These are used relatively for smaller draw-downs.

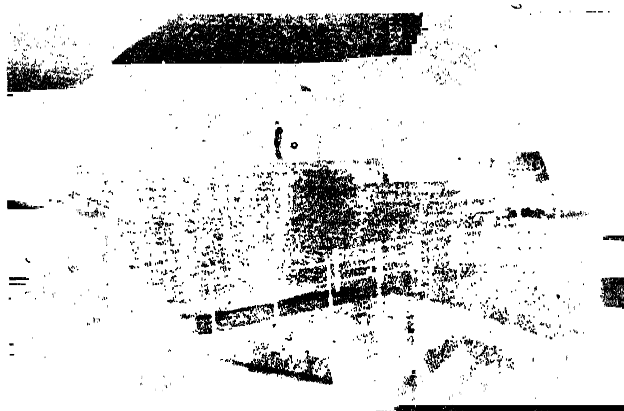


Fig. 29.

Diversion weir, Pykara.

For high head falls the intakes are independent structures, and where overflow *diversion dams* are used, they are generally located at right angles to the dam, thus there is great convenience to clear logs, trees and floating debris by opening one or two

of the nearest flash boards. These are used in general where the draw-down is very considerable. *Maintenance of intake* is just similar to maintenance of the reservoir. Silt which may find lodgment on its bed may be drawn off by a submerged sluice near its terminal.



Fig

Channel clearing in general shut-down once in three years, Shiva Samudram

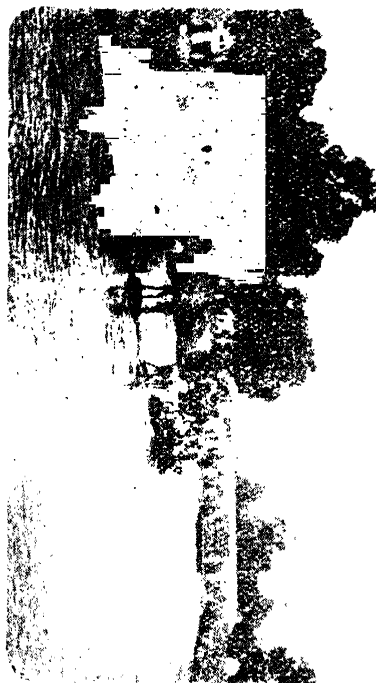


Fig. 30

Clearing of forebay in general shut-down, Shiva Samudram.

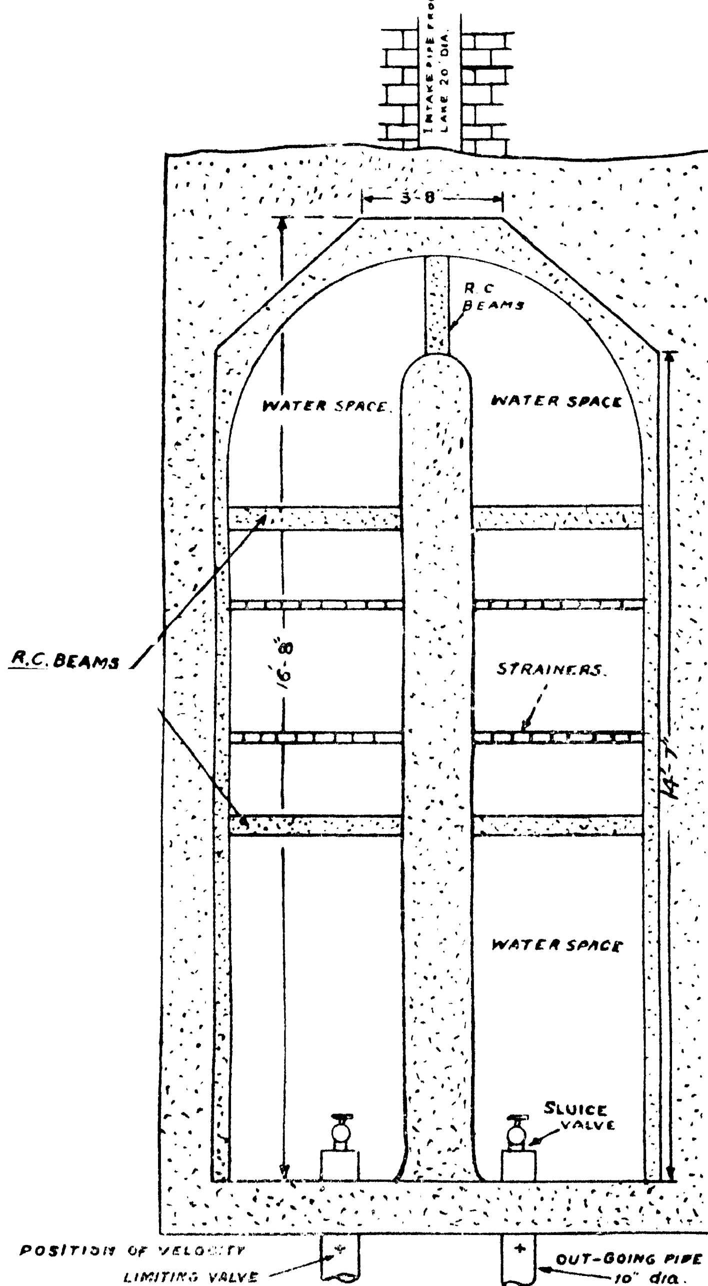
Full velocities in intake gates vary between 2·5 feet per second with an average of 4·0 to 5·0 feet. The entrance to the canal is guarded by *head gates* affording complete and ready control of the inflow. Head gate maintenance involves all that has been mentioned for reservoir sluice gates. They should be frequently tested specially during the winter season. Equipment is likely to be modern and its efficiency is independent of the care and attention of operatives.

Intakes of various types are used according to local conditions. An essential feature common to all types having a bearing on economical use of water is the trash rack—*vide* Fig. 28. They should be so constructed as to give sufficient area for passing the desired quantity of water without excessive loss of head.

In the Nainital intake chamber, Fig. 32, water is carried from the lake in a 20″ diameter C. I. pipe with the invert at the head at R. L. 6855·22 feet to the screening chamber. This chamber

is divided into two parts so that either or both the power pipe lines can be used and is provided with a 22-inch diameter scour pipe. At the head of the power pipe line, Glenfield and Kennedy velocity limiting valves are fitted and have been set to come into action at above 3 feet/sec. velocity. The screening chamber has two screens, each $\frac{3}{8}''$ thick vertical steel, which arrests the entering of weeds, etc., into power pipe lines.

Fig. 32.



Plan of intake chamber, Nainital.

Headworks

Headworks are works at the point where the water is diverted to the canal or pipe line from the main river. Thus it may be the main impounding dam across a river—as in Jhelum—or uses the tail race or spent up water of another plant located higher up on the same or a different stream as in Darjeeling, or it may be the pick-up weir erected across the stream

before the water is taken to the supply channel as in the Gokak installation. Headworks are not necessary if canal is the source of water supply or a dam raises the height of the water and gives an artificial head where there was none. It may consist of a low weir to even up the level of the natural channel and divert the water to where it will be tapped off into a canal or directly into a forebay. In canals, the headworks may consist of a diversion channel at a point where a fall occurs ; and the same may be said of falls on rivers where it may happen that very little in the way of permanent works is required, if the ground is naturally suitable. Gratings are, however, always necessary to prevent timber, ice, or general debris from entering into the operative part of the system ; and controlling gates are needed for regulating the supply and for shutting it off in case of accident, or for repairs or cleaning-out operations. It is also often necessary to carry out a certain amount of river training work to ensure the safety of the headworks proper.

The point at which a river or stream will be tapped, and the manner of tapping it, will depend on the maximum and minimum natural flow, the amount to be diverted, the levels above the headworks, and their geological formation.

Where the head is developed by means of an open channel, it may be sufficient to construct a low weir to level up the bed of the stream without appreciably raising it.

Where the *water is taken directly from the headworks* into a pipe line, it is more than ever necessary to take precautions against foreign matter entering the system. The headworks then become the forebay. On very low heads, dealt with by plant arranged on or inside the dam across the river, the water has to be taken as it comes ; and owing to its low velocity in the wheels, no great harm is done by such matter as is carried through.

In the case of large natural reservoirs the dam itself will generally be the headworks. In such cases the water will not have any silt to affect the turbine, though a reservoir will slowly silt up. In the case of rivers, mud and debris come from the source of supply to the turbines and loss of efficiency soon results from work needles and nozzles due to the action of even the finest grit if the head is, say, 1500 or 2000 ft. Deposition of stones or heavy matter will reduce the capacity of reservoirs and thus they have to be cleaned occasionally. This is expensive. Under sluices may enable the debris to clear itself if there is enough water to spare and if the lake is only of moderate size.

"Where *the headworks have a considerable dam*, there is no danger from stones or heavy matter except that their deposition will reduce the capacity of the lake; but in full torrents this in itself is so serious a matter that high dams would seldom be justified. Such a lake could only be cleaned out at great expense. If, however, such a dam is necessary, it will sometimes pay to put another and smaller one further up stream simply as a means of arresting the silt for a few years. The capital charges would generally be less than the cost of periodical cleaning out. With lower velocity of streams such dams are often practicable though not necessarily at the point where the water is tapped off."*

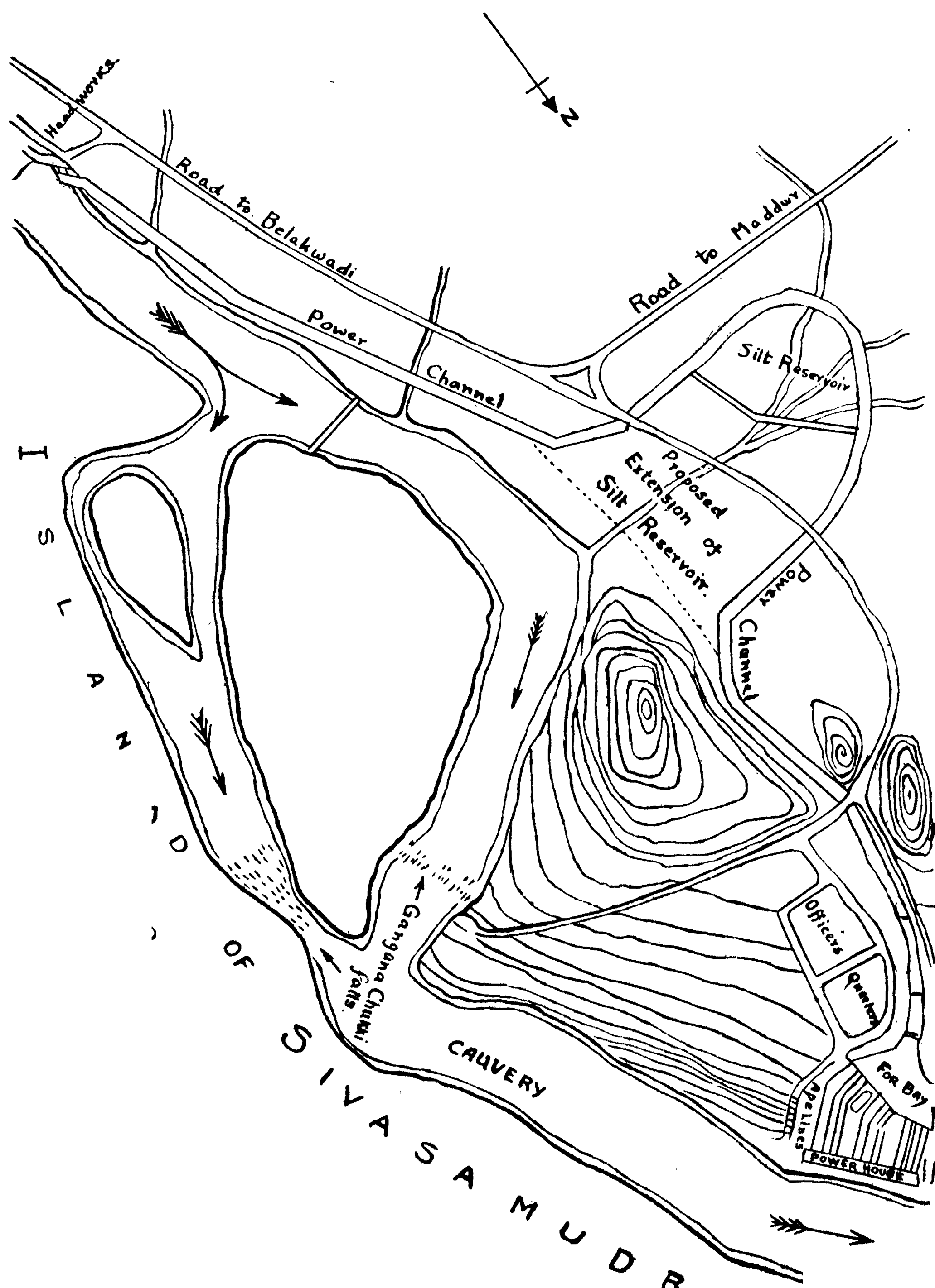
In *low head plants*, the water is taken directly to the flume in which turbine runners are installed, from an opening in the side or corner of the dam as at Jammu.

Cauvery Headworks

As mentioned previously, the Krishna Raja Sagara Dam, which is the chief regulating reservoir of the Cauvery power scheme, is situated about 6 miles from the generating station, Sivasamudram coming down the river, but before the headworks, we have two anicuts—the Dhanegiri and the Madhavamantri,—8 and 20 miles above the falls, respectively. Still coming down the river we have the headworks which, we already stated, is the point where the work of nature ends and that of man begins.

*Adopted from a paper on "The General Principles of the Development and Storage of Water for Electrical Purposes," from the Journal of the Institution of Electrical Engineers, Vol. 57, No. 283, June, 1919.

Fig. 34.


N
Headworks
Road to Belakwadi
Road to Maddur
Power Channel
Silt Reservoir
Proposed Extension of Silt Reservoir
Power Channel
Gangana Chukki falls
Officers Quarters
For Bay
Pipe Lines
POWER HOUSE
CAUVERY
ISLAND OF SIVASAMUDR

A general idea of the river arrangements of the works at Sivasamudram is shown in the map, Fig. 34, from which it will be seen that the river divides into two branches at a distance of 3 miles from the generating station. A low level dam was constructed across the right branch to divert all the flow at low water periods into the left (Northern) branch or Mysore side. During high water periods, this dam has very little influence on the division of water between the two branches. The diversion dam averaging about 8 ft. high and 2,000 ft. long, with the head gates and scouring sluices, is located at about a mile above the falls. They deserve no special description except that in the monsoon period water flows 11 ft. deep over the dam and used to carry away the coping stones. When the dam was raised in 1926 by 3 ft., a continuous layer of ferro-concrete 6″ thick was put on, instead of the coping stones, which has successfully withstood the floods. A general view of the headworks is shown in the Fig. 35 and a sectional view is given in the Fig. 33.

Fig. 35



Headworks and Cauvery River.

The head sluice contains 8 vents $4\frac{1}{2}$ ft. × 5 ft. (in each channel), with lifting shutters, and is flanked by a scouring sluice having four vents of similar dimensions.

Constructional details of the anicuts of Cauvery River :—

Madhavamantri Anicut :—

Total length of anicut—2,870 ft.

(1) Supply sluice—sill 72′, crest level—85·3 ft.
Weir—83·8′, consists of 4 vents—4′ × 5′.

(2) Scouring sluice sill—78·8′, consists of 1 vent—3′ × $4\frac{1}{2}$′.

(3) Head sluice sill—80′, consists of 4 vents 4′ × 4′.

The zero of the gauge post is fixed at 78·8′ and is altered to 72′ on 30th January, 1924.

Dhanegiri Anicut :—

(1) Jagirdar's sluice at right side of anicut—sill = 0. Consists of 3 vents.

(2) Gauge post zero is fixed in connection with zero of the jagirdar's sluice.

(3) New supply sluice.

The sill of the supply sluice is 1′ 11″ below the zero of the jagirdar's sluice and consists of 6 vents—4′ × 5′.

The amount of water to be indented from K. R. S. dam = requirement at Siva + 5% transmission losses.

The programme of letting down water in the power channel is as follows :—

From.	To.	Depth in channel.	Discharge.
3 A. M.	... 3 P. M.	... 8·05′	... 1100 cu. ft.
3 P. M.	... 9 P. M.	... 7.7′	... 1009 cu. ft.
9 P. M.	... 3 A. M. next day	... 6·85′	... 903 cu. ft.

Kollegal Anicut.

Length of anicut ...	... 306′.
R. L. of crest of anicut ...	... 1002·42′.
R. L. of zero of gauge post	... 1002·42′.

Sivasamudram anicut details.

Length	... 2089′.
R. L. of sill of scouring sluice	... 987·5′.
No. of vents and dimensions	... 4 of 4½′ × 5′.
R. L. of sill of head sluice	... 989·00′.
No. of vents and dimensions	... 8 of 4½ 1″ × 5″
R. L. of crest of anicut	... 997·6′.
R. L. of several gauge posts	... 997·6′.

Formula for calculating the river discharge at Madhavamantri and Dhanegiri anicuts :—

$$3{\cdot}1 \times \text{length} \times \sqrt{h^3}.$$

At Sivasamudram :—

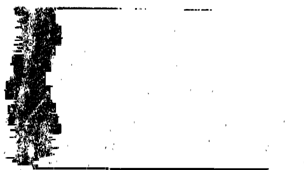
$$3{\cdot}566 \times \text{Length} \sqrt{h^3}.$$

Time taken for water to flow :—

From	To	Time
K. R. S. Dam	... M. M. Anicut	... 2 days.
M. M. Anicut	... Dhanegiri Anicut	... 14 hours.
Dhanegiri	... Headworks at Siva	... 10 hours.
Headworks	... Forebay	... 2¾ hours.

Telephone line along flume
Elevated wall
gate
Stone work bound with clay mortar
Stair Case
Masonry Channel (main flume)
Head works channel
Arched Channel below
Upper gates
gates
Lower gates
10
10
Wooden grating for preventing lags etc.
Spill way Channel
Wooden Stair Case
Stone work bound in wire netting.
Jelum River

Headworks of Kash[illegible]





Headworks, Buniar.

Mahora Power House

Fig 36



Mahora Power-house

Headworks :—The headworks are situated 6½ miles from the power-house and about 2 miles from the village of Rampur. At the headworks, some part of the water of the river Jhelum is diverted, and brought by the flume line, which is a covered wooden channel, to Mahora, where, it is utilized for the generation of power. The arrangement of how the water at the headworks is regulated and allowed to come in the flume is as follows :—

In the bed of the river Jhelum, at the headworks, is a stone bed which ordinarily divides the river into two branches when the water level is normal.

Here, as shown in the figure 28, a net work has been placed at the small mouth of the auxiliary branch. This net work is intended to catch all floating material, such as wood, trees, etc. The net work allows the water to go into the auxiliary branch, which may be called as the water reservoir I. A water level indicator has been fixed up in the reservoir, which indicates the depth of water in the reservoir.

From the reservoir, the water is made to go in a second chamber which is simply a passage to the flume line. The entry of the water therein is regulated by gates. There are 10 gates in all for this purpose. Five of them are upper gates and the other five are lower gates.

Fig. 37.



Gates at the headworks for controlling the discharge into the channel side-view.

When the level of the water in the river is quite high as it is in rains and summer, the upper gates are kept open and the water is allowed to pass through them. The lower gates remain closed. The amount of opening of the gates, and the number of gates open, depends upon the amount of water which it is found necessary to supply to the flume. The control for this is from the power-house by telephone.

The lower gates are usually used in winter, when the flow in the river is minimum and the water level goes down considerably. All the ten gates are hand-operated through simple worm wheel gearing. Further-on, from the second reservoir, the water goes to the flume line and this is regulated by a gate, some four gates have been used as cleaning gates. Their function is as follows :—

Fig. 38.

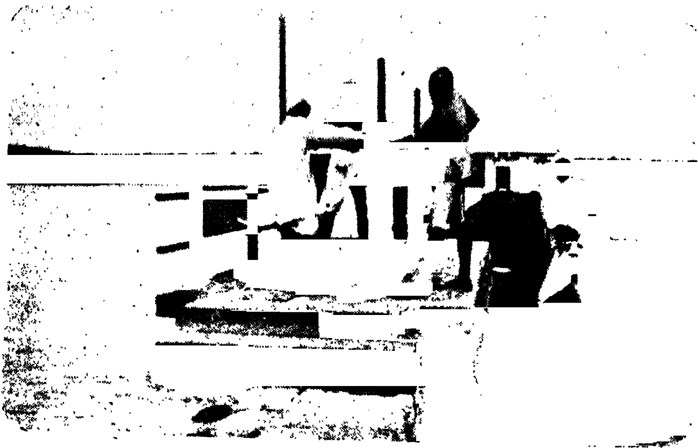


Removing the Silt.

The river is continuously bringing silt and this silt deposits constantly in the reservoir. When the accumulation of the silt becomes much, it affects the depth of the reservoir and the quantity of water in it. So, it is essential to remove this silt and clean the

reservoir. When this is to be done, the cleaning gates are opened and water allowed to go through them to the river. This water takes along with it the silt from the bottom.

Fig. 39.



Operating the sluice gate, Shiva Samudram.

When the flow of the river is considerable, as it is in rains, these gates may be kept always open and a part of water kept continuously running.

The gates are hand-operated and are exactly like the other gates at the headworks.

Jammu Head Race.

Fig. 40.



A view of the power-house at Jammu, showing the water from the turbines joining the main river.

First, we have the escape gate. This gate is used only when the canal is to be cleaned. The canal escape gate goes through a *nala* and joins the river Tawi.

There are four sets of lock gates (each set containing two gates). These gates are hand-operated by worm wheel gear. Each set of gates is directly in front of a turbine pit. The first two gates are smaller and the other two gates are bigger. This is due to the fact that the turbines Nos. 1 and 2 have got alternators of 187·5 kW. capacity coupled to each, while Nos. 3 and 4 have 350 kW. alternators each. Each gate rests in channel section cast iron girders placed in the grooves in the piers.

Next, we come to the overflow gate. This is the gate by means of which the water level in the canal near the power-house is kept constant. By constantly operating this gate, the quantity of overflow water, *i.e.*, water in excess of that required for the running of the turbines, can be regulated. If the water level in the canal goes very low, this gate is completely closed. If there is heavy rush of water, the gate can be opened as desired.

After passing through the overflow gate, the water falls into a well. The thickness of the well masonry has been varied all-round to resist the pressure of the water and that is the reason why the well masonry is much thicker in the centre than in the sides.

The depth of the well is about 28 ft. and its diameter at the top is 12 ft. From the bottom of the well, the overflow water meets the main flow of the tail race and a canal of the same magnitude as before is again formed.

If any of the turbine lock gates give way, then how is the water in that pit to be controlled ? It will be seen in the piers, that there are grooves in them on both sides. Whenever any of the lock gates breaks, then planks of timber of suitable sizes are forced into these grooves one after the other, till the whole thing is closed. Such pieces are always kept in readiness at the power-house. These pieces have grooves cut in them on one side, while on the other side is a projection. The projecting portion of one piece fits into the groove of the other and thus a good joint is secured.

A float is kept in the canal and a wire has been taken from its top to the switchboard and by means of suitable graduations, the water level in the canal is directly indicated on the switchboard.

Headworks, Mandi

The site for headworks is situated just down the village Brat near the confluence of the Uhl River and its tributary, Lumba Dag, approximately at the elevation of 6,000 ft. All the calculations of the headworks and the design of the whole project have been based upon the minimum stream flow which is hoped to be had continuously—(*vide* pp. 20-21). The measurements of the discharge of the Uhl river extended over three years from 1922 to 1926 and the minimum discharge available is 100 cusecs, as recommended by the Engineering Committee, but the calculations are all depending upon 225 cusecs in the 1st stage and 600 cusecs in the 2nd stage.

The scheme consists of 3 developments. The 1st of these, which is now complete, involves the diversion of the Uhl river, by means of a small headwork, settling basins and a diurnal storage reservoir, through $2\frac{1}{2}$ miles of $9\frac{1}{4}$ ft. concrete lined pressure tunnel, from the south end of which water is dropped through a fall of 1800 ft. to a power station situated at Shanan.

The 2nd stage is the extension of the 1st consisting of the provision of a dam and addition pipes and plant, but making use of the same tunnel. The 3rd stage is a separate development utilising the additional head available below the power-house of the 1st and 2nd stages.

Fig. 41.



The double decantation chamber and honey comb, Mandi.

Taking from the very beginning, the project consists of a couple of weirs, diverting the water from the two main tributaries of the Uhl river, the Lumba Dag and the Uhl located at an altitude of R. L. 6100 ft. The flow of Lumba Dag is taken by a small flume and discharged into the Uhl river above its weir, and the combined couple of "coarse and fine screens" for getting rid of logs and boulders through a pair of intake gates into "double decantation chamber," each half containing two chambers in series, designed to remove gravel and coarse sand, respectively. The velocity in the 2nd portion is reduced to one foot per sec. These decantation chambers during the flood season will operate with a continuous under-scour to ensure the continuous removal of the deposited matter. The outlet of the decantation chamber discharges into a high velocity flume approximately 200 ft. long ending into the "blue pool settling pond" from which two 6′-3″ diam. "pressure ducts" 2150 ft. long carry water across a *nala* bed to the upstream end of the "upper main rapid." This drops the water by means of a rapid 190 ft. long through a grade of 1 in 6 through 31 ft. head into "upper settling pond" from which a gate on one side wall will enable 300 cusecs to be discharged at a latter date if required into an "upper emergency storage reservoir." The outlet of the "upper settling pond" discharges into an open flume 200 ft. long. It then passes down another rapid 152 ft. long through a further drop of 37 ft. into the main "forebay" which also acts as a "stilling pond." The forebay is provided with an overflow back into the river, a "discharge gate" and "filling flume" at one side for discharges up to 300 cusecs into the "main diurnal reservoir" during the winter months when the water is clear, a 36″ diam. inlet for discharges up to 100 cusecs from the above-mentioned emergency reservoir and finally the "main screened outlet" into an 8′ diam. "Pressure duct" connecting direct to the main tunnel, for use during summer months when the water is too dirty to be safely stored in "diurnal reservoir," being controlled by 8 ft. diameter "butterfly valve" at the entrance to the tunnel itself.

Artificial Waterways

There is a great variety of types to be used, being entirely governed by the nature of the development as well as by economy. Where the power-house is located near the source of water, there may be no need for conduits at all, as in low-head plants, or they may simply consist of very short pipes. For medium and

high-head developments, however, a more elaborate system of conduits must be provided, as the water must, in many such instances, be diverted for miles, before it finally reaches the power-house.

The different kinds of water conductors in general use may be divided into two classes, open or closed, the closed conductor being either of the low or high-pressure type. The canals and the flumes belong to the first type. The flume may be of wood, concrete or steel. In the next type are tunnels and pipes. The pipes may again be of wood, concrete or steel.

On comparatively low-heads, the waterway may be a canal of large capacity, excavated in earth or rock, and if necessary, lined. Where the required capacity is smaller, and artificial channel of wood, concrete or metal, is employed, either founded on a track on the hill-side or supported on a trestle frame-work, or on pillars. Ordinary masonry is not so satisfactory as concrete, and if used, should be lined with cement.

When the contour of the country is very irregular, the cost of excavation for canals and of building of high trestles for the flumes may be very high. In such instances the closed construction generally becomes more economical in that tunnels may be built and the pipes follow more or less the contour of the country. The selection of the particular conduit is an engineering problem of considerable importance and has to deal with the economic operating features of the development.

When cut through natural earth the velocity of water in a canal is limited to about 3 ft./sec. The necessary flat grade very often involves a tortuous course and in a country of uneven land, the canal will involve heavy earth-works, culverts, etc. In excavating a canal there must be slope to the sides and the angle of the slope must be made slightly greater than the angle of repose of the material through which the cut is made.

The velocity of water in a canal is affected by the roughness of the bed, by the wetted surface of the form of the cross-section and finally by the grade. For calculations vide chapter VI.

The main considerations for the designing of diversion canal.

The canal prism should be of an *area* sufficient to pass the flow at a velocity not exceeding three feet per second; the diameter of pipe conduits depends upon the ratio of head which can be economically expended in friction.

Canals *serve to divert volumes* exceeding 500 cu. sec. *Their design* is based upon the theory of flow in open channels. The velocity in a diversion canal should not be excessive; as a rule,

it will be important to conserve the available head wherever practicable, and the design of the canal affords one of the important opportunities to practise this economy. Only where the excavation of the canal prism presents a very costly undertaking, such as when it has to be located through a hard rock ledge, or in case the value of the necessary right of way is practically prohibitive and when the canal section, therefore, must be kept at a minimum, are high velocities excusable ; five feet per second is a good limit to be adopted for the flow in a diversion canal.

The location should be the economically shortest, which is determined by the cost of the right of way and of the construction, and is most readily proved by the method of elimination,—that is, by making proper locations based upon the survey and boring data and finding the excavation quantities and slope areas for a flow section of one-fifth of the maximum volume to be diverted, thus assuming a trial velocity of five feet. Deviations from a tangent alignment will generally prove justifiable to avoid side-hill cuts, buildings, road crossings, rock out-crops, swamps, and to secure uniformity of the prism, but curvature should be limited to 3°.

The slope in curved channels is greater than in straight ; the excess is determined from Humphrey and Abbott's formula—

$$hc = v^2 \times 6d \div 536\ p.$$

where, v is the mean velocity, d the total angle of the curve expressed in radians ($1° = 0{\cdot}01745$), and $p = 3{\cdot}1415$.

Power Channel

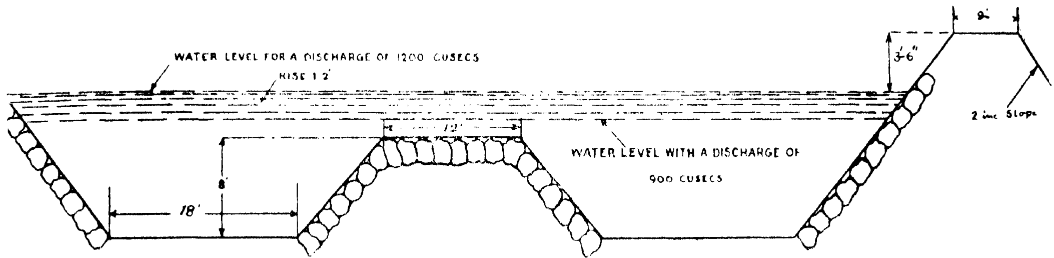
Siva Samudram

Channel :—From the head gates two canals follow the best line to the forebay and at present both form a single unit.

The power channel has been designed to carry 1200 cusecs at full load capacity of 37,500 kilowatts. Admission of the water to the channel is regulated at the headworks by hand-operated sluice gates by a gauge man whose duty is to regulate the gates according to the instructions he receives from the generating station through a telephone—*vide* Fig. 37. During the earlier part of the project when the capacity of the station was considerably low, there were two channels side by side of equal capacity to meet the demand through either, in the case of the other having to be emptied for cleaning. But the latter expansion of the scheme

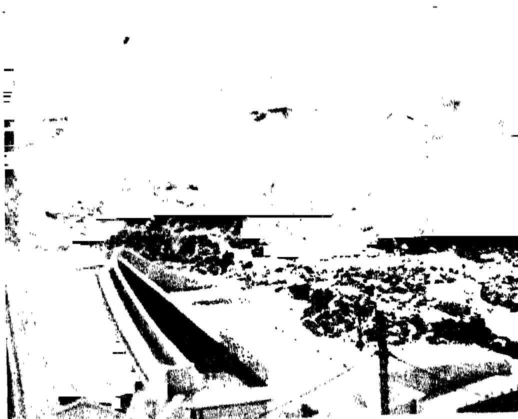
has necessitated the amalgamation of both into one which, as it is carries at peak load of station about 1000 cusecs, *i. e.*, 30,000 kW.

Fig. 42.



Looking to Fig. 42, showing a cross-sectional view of the channel, it will be noticed that both the channels have got trapezoidal section and have their sloping sides built with stones. At present both form a single unit, the central earth bank being submerged. The channel terminates into the forebay bringing the water from the headworks. The length of the channel from the headworks to the forebay is 3 miles, 2 furlongs, 440 feet. The maximum capacity of the channel is 1200 cusecs. The grades of the channel are 1/5000 for rock sections ; the bed widths being 18 ft. and 12 ft., respectively, with stone pitched side slopes of 1/1. The normal depth of water is from 10 to 12 ft. Ample provision has been allowed in the way of aqueducts (under which surface drainage is *carried*) and also *escape weirs* in the banks to meet emergencies. On the third mile from the headworks the channels run through intervening high ground that entailed heavy cutting from 20 to 34 ft. in depth for a length of 1200 ft. in hard blue rock. Here the partition between the channels is formed by a wall of rubble masonry, the sides of the

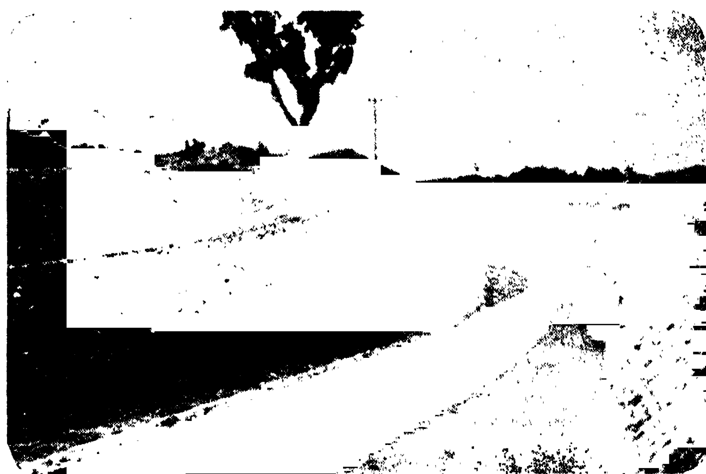
Fig. 43



The escape weir and water discharge.

cutting being also vertical. The quantity of dynamite expended in blasting operations on this section alone was nearly 8 tons. At present about 1,000 cusecs are required. Within a distance of 3 furlongs from the forebay there are constructed three "drops" of 6′ to 6½′ each. The object of these small artificial drops is to reduce the velocity of the channel water to a desired value. The following table gives the location of various aqueducts, waste weirs, relieving weirs, etc., along the length of the channel.

Fig. 44.



Open canal through deep cutting.

Distance from Headworks.			Name of the object.
Miles.	Furlongs.	Feet.	
0	6	486	Culvert No. 1 for the road-crossing channel.
0	0	601	Aqueduct No. 1 with two vents of 6′ × 5′ each. Also a waste weir at left to a length of 31 ft.
1	0	0	Relieving weir No. 2 at right channel for a length of 50 ft.
1	2	440	Culvert No. 2.
2	1	322	„ No. 3.
2	5	628	„ No. 4.
3	0	217	
3	1	0	Drop No. 1 of 6 ft. fall.
3	2	140	„ „ 2 „ „ „
3	2	440	„ „ „ „ 7 ft. „
			Forebay.

Ranbir Canal

The canal which has been utilized for the production of power is the Ranbir canal, which was dug out for irrigation purposes about half a century ago. It gets its water from the river Chenub. Throughout its length in Jammu, some sub-canals have been taken out of this to distribute the water throughout the land, the main object of the canal being to irrigate the lands. The average width of the canal is about 30 ft. and the average depth of water in it is about 5 ft. The ground being very undulating, the canal, in many places, had to be carried over via ducts, and there are aqueducts also along its route where the canal water goes through subterranean passages under ground, such as below the bottom of a river which it has to cross at a difirent level.

The power-house has been erected at a place about 18 miles from the headworks, from where the canal flows out from the river Chenub. The water in the canal is rather dirty all through the year and during the rains much silt is carried down by it. The Chenub being a snow-fed river, the water is very cold throughout the year.

During winter the level of the water in the Chenub falls very low, because the melting of the snow ceases in the mountains. The Ranbir canal is heavily drained during the winter season, say from the middle of October, that being the cultivating season, and this is also partly responsible for the falling off of the level of water in the canal during that time. The quantity of water flowing through the canal is so much lessened during the winter that the power supply from the electric installation becomes quite inadequate for the town of Jammu.

The velocity of water in the canal is about 8 ft./second near the power-house during the summer season. During this season almost the whole of the water in the canal is passed through the turbines at night, the rest being allowed to go to the tail race canal directly through a well near the power-house. During the day-time when the load on the power-house is low, about half the quantity is only required for the generation of power. For preventing the flooding of the power-house buildings during heavy rains, there is an over-flow channel with a sluice gate to by-pass the extra water clear out of the power-house to the river Tawi just near-by.

Flume Line

A flume is generally wholly or partially supported on timber or steel trestles or masonry benches. The support foundations

are responsible for the permanency of the shape of the flume and therefore largely for defects causing leaks. Flumes must be kept free from ice to prevent rupture and should be provided with frequent spill-ways or sluices for ice passage.

Excavation cost depends upon the character of the material, the quantities to be moved, and the disposition which may be made of the spoils ; all these, together with the depth and width of the cut, will influence the methods which secure the most economical excavation. Rock is most cheaply removed in the dry season. The principal operations involved in excavating hard rock, a classification which includes those formations which can be excavated in large masses only by the use of explosives, are drilling, blasting, loading, and disposing.

Rock-drilling may be done by hand tools or by machine drills operated by steam or compressed air; only machine-drilling will be here considered. This operation requires a runner and helper, power being supplied from a central plant. The output varies with the depths of the holes, being from 50 to 60 linear feet for 10 and 20 feet depths per shift of 10 hours. To the operating cost must be added the power cost for 10 horse-power per drill, repairs and drill sharpening; with present wages drilling costs from annas 8 to Rs. 3 per linear foot, depending on the hardness of the rock and the depth of the holes.

The breaking of the rock for the purpose of loading for removal requires from 1 to 1·5 pounds of 60 per cent. dynamite, according to its hardness, per cubic yard of output, and with present cost of explosives a charge of 8 annas per cubic yard should be made for blasting. The broken rock may be loaded by hand, or machinery, into carts derrick or cable-way buckets, or dump cars, and the cost varies in accordance with the method employed; the output of hand loading averages 10 cubic yards per man per 10-hour shift, by steam shovel it will be between 40 and 60 cubic yards per hour. *Disposal cost* depends entirely upon the length of haul. With present wages rock excavation will cost from Rs. 3 to 1·50 anna per cubic yard.

Appertinent structures to diversion canals are those required for the safeguarding of the superbanks against erosion from surface run-off, interception of unavoidable lateral stream sources, the devices controlling the flow in the canal, and means of overhead crossings.

Pipes are the last type of diversion conduits and are then known as Pressure Lines.

The Flume-Line at Mahora & its Reconstruction Work.

In the Mahora hydro-electric installation, the power-house is about 6½ miles from the headworks and the fall of the river in this distance is about 500 ft., as mentioned previously. Owing to the unevenness of the country a canal was out of the question. The soil was very hard and difficult to excavate and timber being very cheap in the locality which abounds with pine forests, a wooden flume was designed to carry the water through 6½ miles. The first cost of this wooden flume was low, whereas if steel flume was used that would have been very high.

Flume Line is a wood section passage to carry water from headworks to the forebay. The flume is 10 ft. by 8 ft. and is 6½ miles long. The material used for the construction of the flume is timber (Deodar), which is in abundance in the locality. It would have been much more troublesome and costly to use any other material instead. But timber has got its own drawbacks. Unless the water is running all the time, warping is certain to occur, and boring insects and rot has to be faced. The flume for the installation suffers severely in this way.

Fig. 45.



A portion of the wooden flume passing through a tunnel.

From silt basin onwards the flume has to pass through 5 tunnels and passages over a number of bridges. Tunnelling has been restored to, to avoid bad ground. Besides this, the bluffs have also been tunnelled to pass the flume line as it has been found cheaper to tunnel than to take the flume round it.

The flume has got a slope of 8 ft. in a mile. It is found that the erection of the flume was quite a difficult job. It is clear from the diagram that the mountain ridge had to be cut and

the plane ground of sufficient width prepared before the flume could be erected. Tunnelling and bridging at many places has made the work still more difficult.

Dangers to the Flume

(1) It is quite probable that some big rock may slide from the hill-side and fall on the flume line. These pieces injure the flume many times, but at times break it asunder completely.

(2) Land slips are very dangerous to the flume.

(3) The flume has to cross local hilly streams and rivulets at many places. It is essential that the flume should be protected from the wash-outs in the rainy season, when the streams run at a very great speed and the quantity of water flowing is also large.

When the water supply in the flume is less, the rivulets in cases can be used for supplying water to the flume. This is only necessary when there is a leakage or breakage.

As far as possible the flume has been constructed to withstand the rolling of stones and land-slips. Inclined planks are placed between the rocks and the flume line. They detain the big pieces from directly reaching the flume.

Sufficient number of planks are always kept in store to be used in case of emergency. The planks have got slots on both sides, they are placed one over the other length-wise and the wedge is thrust between the two.

Fig. 46.



Spill-ways in the wooden flume, Mahora.

The size of the flume line is sufficient to carry water capable of generating 14,500 kWs. in the power-house cotinuously, but as the installation is only 4,000 kW., the depth of the water in the flume is usually kept 3 ft. only.

Along the hill-side there are many fields and the people live there ; flume water is supplied to them for irrigation and for personal use.

Overflow of the water of the flume can also be adjusted by the drains fitted to the body of the flume line—*vide* Fig. 46.

However, a keen and constant eye should be kept over the flume line. Wash-offs, breaks, leakages and land-slips are most common.

To keep strict watch over the whole line of 6½ miles long, 'chawkidars' or guards are kept to *patrol* the whole line day and night by shifts. But it is necessary to check and judge the amount of work done by the guards.

All along the flume at certain intervals a key is fixed to the body of the flume line.

Every man going for the patrol is provided with a patrol watch. It has a paper movable dial. The shift engineer puts a new dial on winds and locks the watch with a particular key in his possession.

The keys fixed to the flume line have got punching pins; they have one, two or three points in consecutive order and repeat it to the end. When a man reaches a key, he puts the watch over the key, which punches the paper dial in as many points as it has got.

A perusal of the dial shows that it has six circles all round the numerals. The first key will punch one hole on the first line, the second will punch on the second line, the third on the third, while the fourth will punch one hole in the fourth line. All these will be punched at the particular time when the guard reaches the key.

When the guard finishes his shift, he goes to the power-house and hands over the watch to the shift engineer.

By the position of the punches the engineer can know, whether the man was slack or quick at work, and the time taken by the man in going from one key to another can well be judged.

However, the lines on the dial are not properly made use of. The guards are instructed to report the condition of the flume line.

The patrol continues all the twenty-four hours all round the year. The job is very difficult in winter, when the Himalayas are covered with thick snows. Rainy season is equally a bad weather for patrolling, and perfect hillmen are elected for this job.

The existing flume is stated to have a capacity well over 500 cusecs at full depth of flow, which is 8 ft. and with a velocity slightly in excess of 8 ft./sec.

The slope of the flume is 1·05 in 1000, *i. e.*, 1 in 970.

$$\text{Hydraulic radius} = \frac{(8' \cdot 4'') \times 8'}{8' + (8' - 4'') + 8'}$$

$$= 3 \text{ ft.}$$

$$\text{Kutter's formula } c = \frac{\frac{1 \cdot 811}{\eta} + 41 \cdot 66 + \frac{0 \cdot 00281}{S}}{1 + \frac{\eta}{\sqrt{R}}\left(41 \cdot 65 + \frac{0 \cdot 00281}{S}\right)}$$

$$\text{Bazin's formula } e = \frac{87}{0 \cdot 552 + \frac{m}{\sqrt{r}}}$$, where $m = 1 \cdot 3$ for earth canals and $0 \cdot 06$ to $0 \cdot 16$ in flume of wood.

η in the Kutter's formula being $= 0 \cdot 012$,

where η is the coefficient for wooden flumes of roughness.

$$\text{We get } c = \frac{\frac{1 \cdot 811}{\cdot 012} + 41 \cdot 65 + \frac{\cdot 00281}{\frac{1}{970}}}{1 + \frac{\cdot 012}{\sqrt{3}}\left(41 \cdot 65 + \frac{\cdot 00281}{\frac{1}{970}}\right)}$$

$$= 151.$$

Whence $v = c\sqrt{R.S.} = 151\sqrt{3 \times 1/970} = 8 \cdot 3$ ft./sec.

The velocity found by actual observation and measurement is slightly over 5 ft./second against the calculated velocity of 8·3 ft. per second. This is due to the enormous deposit of debris on the floor of the flume scattered all over its length and also due to the numerous sharp-bends in its course. Accordingly, the discharge is not more than 313 cusecs, at 7 ft. 6 inches depth.

The flume-line was constructed in 1906 and so is 25 years old. The first sign of decay and rotting of the Deodar wood scantlings was noticed about 8 years ago.

In 1924, a partial relining of the floor and the two sides with Deodar wood planks was begun and the work was completed

in 1926. After about a year or so it was noticed that practically no more useful life was left in the flume and it was not possible to keep it intact by repairing portions of it. The complete renewal of the planks was found to become a necessity for the continuity of the power supply. The bottom portion of the flume was found to be so badly rotten in many places that it might give way at any moment. But for constant watch and timely patching here and there, this would really have collapsed years ago.

In the same year it was considered that if this wooden flume could be replaced by steel 'Armco' flumes, there would be very little trouble for about 40 years.

The 'Armco' flume is a steel flume of semicircular section. The plates are rolled with a bend at each end so as to form an interlocking joint. It is made water-tight by means of a curved rod fitting the outside of the groove and a curved bevelled bar on the inside. They are supported on timber frame-work.

The following were the arguments for and against adopting the 'Armco' flume :—

(1) It is mechanically correct.
(2) It is water-tight.
(3) The smooth interior ensures the greatest carrying capacity.
(4) The joints care for expansion and contraction.
(5) It is easily transportable and easily erected.
(6) Maintenance cost is much less than the wooden flume.
(7) Changes in the season have not got any effect on it.
(8) They are not permanently damaged by any setting of the foundation.
(9) Its life can easily be 40 years.

Deodar wood flumes last for about 20 years, but the scantlings can easily be replaced without disturbing the flume or the water supply. To carry the same quantity of water, *i.e.*, 300 cusecs, for which the existing wooden flume was designed, the top over-all width of the 'Armco' flume should be at least 15 ft. against 12 ft. 6 of the wooden flume. This is a great drawback as in some places the wooden flume is built on narrow benching not more than 12 ft. 6 inches in width cut right in the rock and to have a wider flume either very expensive substructure or rock-cutting would be necessary.

The renewal of "Armco" flume in the case of any accidental break due to land-slip, etc., would be a very simple matter and could be done very quickly provided the necessary material

was ready at hand, whereas rebuilding the existing flume takes a long time, even when the necessary grooved and tongued planks, sills, posts, caps, etc., were ready.

The cost of the 'Armco' flume is, however, very heavy. Taking the transportation charges and the erection costs into consideration the total cost per foot run comes to about 53 rupees. 4·67 miles of 'Armco' flume would, therefore, have cost Rs. 13,06,821, to which was to be added the cost of housing the labour which is about Rs. 50,000, thus bringing the total to Rs 13,56,000.

The estimate for the complete renewal of the wooden flume amounted to Rs. 6,22,400. The reconstructed wooden flume would give a discharge of 338 cusecs with a velocity of 6·5 ft. per second as found by the following calculation :—

The slope as actually found by levelling was 1 in 920 and hydraulic radius 2·5 ft.

$$R = \frac{8' \times (6' - 6'') \text{ allowing } 6'' \text{ for the free-board}}{(6' - 6'') + (6' + 6'') + 8'}$$

$$= 2{\cdot}5 \text{ ft.}$$

η = ·04 in Kutter's formula.

Even if 300 cusecs is taken as the discharge allowing 38 cusecs for leakage and obstruction in the flume due to the falling debris, it is more than that of the proposed 'Armco' flume which was about 270 cusecs. It was decided not to take up the 'Armco' flume as it would have cost a great deal.

The flume reconstruction work was begun in 1927 and in 1928, 358 ft. length of it had been constructed.

Fig 47.



Flume Reconstruction, Mahora.

The reconstruction work is still going on and it is only in Sundays that the work can be carried out. The water is shut down in the headworks for about 20 hours and about 60 ft. length of the flume is pulled down and reconstructed in the period of 20 hours. The water of a nearly streamlet, called the Mahora Nala, which is used to supplement the flume water, is then used alone to run one of the generators.

and an axciter. This prevents a complete shutdown of the power-house.

All the necessary planks, scantlings, posts are kept ready, the saw mill in the power-house compound being kept busy throughout the week for preparing them.

The water in the flume runs about 3 ft. deep now-a-days as this amount is sufficient to supply the 4 sets installed up till now. At about 12 in the night on Saturdays the sluice gates at the headworks are closed and the water in the flume drains off in about 2 hours. The reconstruction work begins at about 2 A.M. with the pulling down of the selected portion of the flume. About 300 workmen are assembled for the purpose, including the carpenters, overseers and coolies. In most places the flume runs on a levelled benching on the sloping side of the hill, and the log and planks of the old flume are allowed to fall down the hill, and this simplifies matters. The floor of the bench is then repaired, where necessary, and made flat. The slope of the bed is then found out by levelling and special care is taken to keep it 1·05 in 1000 all along.

The floor is rammed in with $\frac{3}{4}$ inch of metalling and over that two planks are placed side by side to serve as mudsills, so that the weight of the flume may not make it sink down in the loose earth. The bottom sills are then placed crosswise at regular intervals of 3′-2″ inches over the two planks. These sills are the bottom pieces of the vertical frame work of the flume. The floor of the flume is formed by nailing planks into the bottom sills. The two vertical posts of the frame are then fixed by mortise and tenon joint to the top horizontal cap and the three are then raised to the vertical position on the corresponding sill. The vertical posts are joined to the bottom piece by mortise and tenon joint.

In the beginning these rectangular frames are kept in their positions vertically by nailing temporarily strips of planks on the posts. As the work of planking up the floor and the sides proceeds on, these strips of wood are removed one by one. The floor is planked up first. The planks for the floor and the sides are 10′ long each, $9\frac{1}{2}''$ wide and $2\frac{3}{4}''$ thick. The sides have got grooves formed in them $\frac{3}{4}''$ wide and $\frac{1}{2}''$ deep. When placed side by side, the grooves in the adjacent planks form a rectangular hole $1'' \times \frac{3}{4}''$ and in these holes are placed tongues of sound Deodar wood to make the joints water-tight. These tongues are of the same length as the planks.

When the floor has been completed, the sides are planked up beginning from the bottom. At the corners are nailed in

triangular fillers of 3″ sides, and jute packing is used to make the corner joints absolutely water-tight. These fillers run continuously along the length of the flume. A tongue is placed between each two adjacent planks on the sides.

As the lengths of the planks are always multiples of the pitch of the framings, the ends of the former always come on the posts or bottom sills, of the frames.

Some of the old planks are then nailed down on the top of the caps to serve as a partial cover and also as a foot path.

To strengthen the structure, struts are placed in the corners, as shown in the sketch. For very sharp curves the sides of the flume are made of three layers of planks each $\frac{3}{4}$″ thick.

The original cost of building the flume line was Rs. 15,90,800 and the estimated cost of rebuilding it is Rs. 6, 22, 400. The cost of renewing every chain-length, *i. e.*, 100 ft. of it, is Rs. 2,524, as per details given below :—

No.	Quantity.	Particulars.	Rate.	Amount.
1.	301·4 cu. ft. ...	31 Deodar wood sills 14′ × 10″ × 10″ ...	-/12-per cu. ft. ...	Rs. 226/-
2.	348·75 ,, ...	62 D. W. posts 9′ × 10″ × 9″ ...	,, ...	262/-
3.	193·75 ,, ...	31 D. W. caps $12\frac{1}{2}$′ × 9″ × 8″...	,, ...	145/-
4.	$\frac{843·9}{10}$,, ...	Due to loss in 1·2 & 3 ...	,, ...	63/-
5.	58·1 ...	62 old D. W. struts each 5′ × 9″ × 10″ ...	From old flume, ...	
6.	29 ,, ...	62 old D. W. ties $2\frac{1}{2}$′ × 9″ × 3″	do.	
7.	87 ,, ...	Sawing and landing charges for struts & ties. ...	Rs. 3/- ...	16/-
8.	562 ,, ...	310 planed grooved D. W. planks 10′ × $9\frac{1}{2}$″ × $2\frac{3}{4}$″	Rs. 1/2/- ...	632/-
9.	13·5 ,, ...	260 D. W. planed tongues 10′ × 1″ × $\frac{3}{4}$″ ...	,, ...	15/-
10.	6·25 ,, ...	D. W. triangular fillets ...	,, ...	7/-
11.	$\frac{581·75}{4}$,, ...	Due to losses in 8, 10 ...	,, ...	163/-
12.	162·28 ,, ...	Do. sawing & handling ...	-/2/6 ...	26/-
13.	163·28 ,, ...	Top plankings of old D.W. ..	... ...	...
14.	$1\frac{1}{4}$ cwt. ...	Talen nails ...	Rs. 38/- cwt. ..	48/-
15.	4 gallons ...	Coal tar ...	1/8/- gallon ...	6/-
16.	$3\frac{3}{4}$ cwt. ...	Wire nails. ..	Rs. 22/- cwt. ...	83/-
17.	5 gallons ...	Soligrum paint ...	,, 4/- gallon ...	20/-
18.	150 cu. ft. .	$\frac{3}{4}$″ stone metalling. ...	6/- 100 cu. ft. ...	9/-
19.	1685 cu. ft....	Carriage of timber from saw mill. ...	-/1/- cu. ft. ..	103/-
20.	1 chain ...	Flume labours construction	... ...	500/-
21.	,, ...	Dismantling old flume & repairing bench ...	...	80/-

Rs. 2,404/-
Contingencies 5 °/o ... Rs. 120/-
Total ... Rs. 2,524/-

This is the cost of renewing one chain of the flume of section 8′-4″ inches wide and 8 ft. high. This is the size of the flume in the lower reaches for 39 chains. The forebay water heads up to 8 ft. in the adjacent masonry channel and in the lower reaches of the flume line and then gradually falls down to the normal level of 3 ft. 6 inches for the remaining length of the flume. Considering the waste of money in constructing the flume so high, the use of which was never required for the last 25 years, the size of the flume in the present design in the upper reaches has been kept at 8 ft. wide and 7 ft. high and sizes of the scantlings too have been reduced without in any way impairing the strength of the flume. The flume was originally constructed for 20,000 h. p., but there is no possibility of having any more plants installed in the power-house in the near future.

The cost of renewing the flume of 8′×7′ section is estimated to be Rs. 2,235 per 100 ft. length and the cost for 202 chains of this would be Rs. 4,55,106 and that of 39 chains of 8′-4″×8′ section would be Rs. 98,436. The total would come, therefore, to Rs. 5,53,542.

The total length of the channel is 6·5 miles or 34,222 ft., consisting of :—

Masonry channel	8,320 ft.
Wooden flume	24,666 ft.
Water bearing rock tunnel	636 ft.
Length of silt basin	600 ft.
Total ...	34,222 ft.

Main Aqueduct, Pykara:—Generally open canals and flumes are used to convey water from the point of diversion to the forebay. A higher efficiency will be obtained by a closed system of tunnels and pipes in that the total head will be greater but in any choice the economic side of it must be correctly considered.

The velocity of water in a canal, according to Chezy's formula, is :—

$$V = C\sqrt{R.S}, \text{ where}$$

$$R = \frac{\text{Area of cross-sec.}}{\text{wetted perimeter}}$$

V = Velocity in ft. per sec.

C = Constant coefficient.

R = Hydraulic radius.

s = Grade of hydraulic slope.

The value C is computed from Kutter's formula or Bazin's formula, which is :—

$$C = \frac{157 \cdot 6}{1 + \frac{r}{\sqrt{R}}},$$

See Ch. VI, p. 150.

For an open canal the grade or slope is the ratio of the fall to the length in which the fall occurs. For a closed pipe under pressure it is the ratio between the loss in head due to friction to the length.

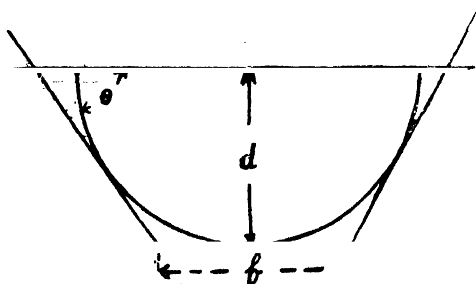
The velocity of water in a canal should be kept large enough to prevent vegetable growth from forming and silt from being deposited. Also it should be kept below that which would cause erosion of the bed.

Mean volocities for concrete	... 15 to 20 ft. per sec.	
Hard rock ...	... 13	,,
Stratified rock ...	... 8	,,

The most advantageous cross-section to use from the hydraulic point of view would be that which gives the smallest wetted perimeter or the largest hydraulic radius. This would mean a semicircle, but such sections are seldom used on account of the difficulties in building and the expense so incurred. A trapezoidal section is however generally used and by letting the bottom and sides be tangents to an inscribed semicircle, the best results are obtained. The two sloping sides will be inclined at an angle θ to the base where $\theta = 60°$.

The flume channel for the Glen Morgan Scheme is of semicircular section wherever ARMCO sections are used and of the trapezoidal section for the open canal length. As mentioned elsewhere, the whole length of the aqueduct is about 2100′, of which there is an open canal section for about 1000′. An ARMCO flume for about another 1000′, and a small tunnel section. Wherever excavations or filling or cutting was necessary ARMCO section was conveniently used so that it could be used elsewhere even after the scheme would be rejected. An ARMCO section is nothing but a wooden or steel structure—wooden in this case as the

Fig. 48.



scheme is temporary—supporting a semicircular pipe conveying water. Even in permanent schemes it is used where costly cutting or filling up will be required.

Ducts of Tata

The ducts have a water capacity of 120,000 h. p., the maximum velocity of water being approximately 5 ft./1 sec.

The country through which ducts pass is very difficult necessitating the construction of 22 aqueducts made up of arches as follows.

No. of Arches.	Span.
3	6 ft.
14	19 ,,
11	14 ,,
3	12 ,, 3 ins.
27	20 ,,
24	30 ,,
3	32 ,,

There are three tunnels cut through rock one being 750 ft. in length, one 280 ft. and another 130 ft., making in all a length of 1160 ft. In addition there are 10 cut and cover tunnels having a combined length of 2630 ft.

The duct walls are built of rubble masonry throughout lined with concrete blocks and the bed is formed of lime concrete surfaced with a fine coating of the same material. In many places extensive protection works have had to be constructed. These consist chiefly of concrete arches thrown over the duct so that in the event of any earth falling from above no damage is done to the duct.

The Lonawala duct joins the Walwhan duct at a distance of about 530 yards from the Lonawala sluices. There are stop gates in each duct near the junction and in the common duct near the forebay. The length of duct from Walwhan to forebay is 7630 yards, this with the Lonawala duct makes a total length of 8160 yds. (4·6 miles).

Tunnels

Where the proposed route of the waterway encounters mountain ridges it is often advantageous to go through these by means of tunnels rather than to excavate deep cuts or go around. The deciding factors are (1) first cost, (2) maintenance.

Tunnels are practicable in any size larger than about 4 by 6 ft. and are the most permanent form of construction. They

may be used under any head if they are deep enough in the ground so that the weight of the over-burden is sufficient to balance the internal pressure, and they are sometimes reinforced with steel where the depth is insufficient.

Loading :—Internal loading or maximum internal pressure in closed conduit.

Merit :—Tunnels are safer and their up-keep is usually lower than open canals.

Disadvantage :—They are very costly.

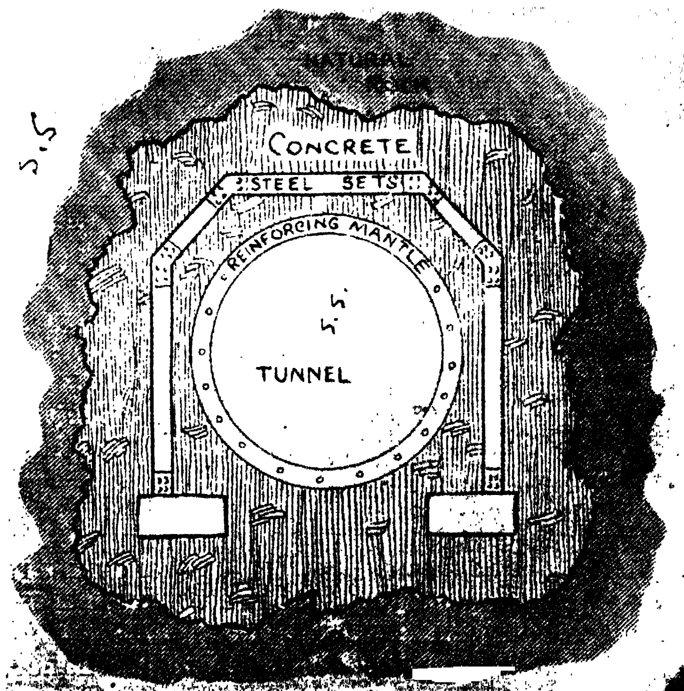
Types :—

(1) Pressure type—when of considerable length.

(2) Non-pressure type.

Thickness of lining varies from 4 to 12 inches according to the grade and pressure of the water and size of the tunnel. The average thickness of lining of rock tunnels is required by practical considerations to be at least about 12 inches and averages 16 to 20 inches.

Fig 49.



Vertical section of a tunnel.

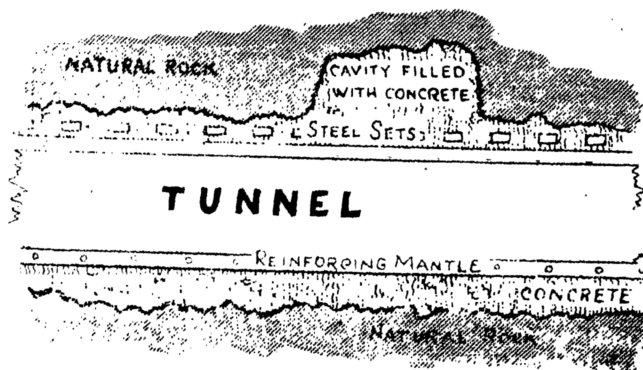
Object of Lining :—(1) To hold the rocky material in place and thus to increase the structural safety of the tunnel. (2) It prevents seepage if the rock is porous. (3) To obtain a smooth surface it decreases the friction and thus permits a higher velocity with a correspondingly decreased section and a minimum loss of head which may be obtained from Kutter's formula in which n may be taken as 0·014 for

lined and 0·028 for unlined tunnel. The safe velocity is from 10 to 15 feet.

Strength of lining :—The lining must be designed to withstand all external water pressure or else drained.

The shape of the section must be such that the lining affords the best resistance to external pressure.

Fig. 50.



Longitudinal section of a tunnel.

Tunnels through very soft earth and earth-tunnels subjected to unbalanced water pressure must be circular or nearly so.

Shape or cross-section :—Circular cross-section would be the most advantageous from hydraulic point of view, but the horse-shoe shape with an inverted arch bottom has been found to be easiest to excavate and is in common use for moderate external pressure.

Tunnels reinforced for internal pressure must be of a shape that will permit of circular reinforcement, but the outside and inside of a lining may vary from a circle for the sake of economical construction, provided the circular reinforcement is well-imbedded.

In long tunnels, adits or openings are usually provided at certain intervals so that the work can proceed from several headings at the same time, thus saving the cost of hauling the much longer distances underground.

Roof of the tunnel is almost invariably supported by a semicircular arch.

The bottom of an earth tunnel must be designed to support the weight of the tunnel and the top load, without settlement. It must therefore be very thick or shaped as an inverted arch to distribute the side wall loads over the whole base.

Height :—To provide reasonable room for workmen and machinery a height of excavation of 6 or 7 ft. must be adopted.

Depth of the tunnel:—The tunnel must be made below faulty or broken or disintegrated area or else to miss them by being located to one side. With tunnel operating under pressure it is necessary to go to such a depth that the weight of the rock above the top of the tunnel is sufficient to counterbalance the hydrostatic pressure.

Adits or drifts in the hill-side are merely side tunnels driven from the face of the hill to the alignment of the tunnel to permit of starting additional headings.

Ventilation:—The ventilation of a tunnel during construction is very important. A great deal of inefficiency results from the lack of proper ventilation. Allow plenty of fresh air from outside at the heading. A fan located on top, near the shaft, generally supplies larger quantity of air at low pressure to a supply main. The ventilating pipe, which is usually 16 inches or more in diameter, is conducted along the tunnel to the heading and is frequently attached to the roof in order to be out of the way.

Tunnel—Tata Power Company's Hydro-Electric Project

The works in this section apart from the tunnel proper included the approach cut, sluice gates and surge shaft. The approach channel as originally cut runs parallel to the old river bed to the sluice gates and is 2,550 ft. long with a slope of 1½ in 1,000. Its average width is 25 ft. and depth 3 ft., the sides being pitched to a slope of 1 and 2 and capped with a dwarf wall in cement mortar 2 ft. high, where necessary. As the bed was rock throughout, no lining was necessary.

Where the old river bed is to be crossed to make a junction with the sluice chamber, a semicircular tunnel 23 ft. in diameter and 265 ft. long was driven under the river bed. This facilitated construction and ensures the approach cut in operation relatively free from debris brought down by the river at the break of the monsoon. As the progress on the dam during the 1925-26 season would not have allowed the first sets to be operated in 1927, a gullet 10 ft. wide and 5 ft. deep was cut in the bed of the approach which was extended a further 1,500 ft. with a slope of 1 in 100. This auxiliary approach cut enters the tunnel proper under one of the main gates through a supplementary tunnel and is controlled by an auxiliary gate.

At the junction of the approach cut and the tunnel proper is a gate chamber in which there are three *Glenfield Kennedy cast iron single faced sluice gates*. Each gate is 8′-6″ in span

EAST

Horizontal Scale 1 Inch 600 Feet
Feet 200 100 0 200 400 600 Feet

Vertical Scale 1 Inch 50 Feet
Feet 50 0 50 100 150 Feet

SHAFT No 1

Plug left here and taken out after 1925 Monsoon

1982 Feet
Fall 1 in 260

SHAFT No 2

Sump

2925 Feet
Fall 1 in 250

SHAFT No 3A

SHAFT No 3

C L of Tunnel

R L 1897 60 (Invert)

PLAN

18" Concrete Gunned Lining

10 Dia

SHAFT No 1 (NOT BLOCKED)

18 Concrete Gunned Lining

Rectangular 12 x 6

SHAFT No 2 (BLOCKED & LINED)

Masonry Cement Pointed

SHAFT No 3A (NOT BLOCKED)

18 Concrete Gunned Lining

SHAFT No 3 (NOT BLOCKED)

SHAFT No 4 (BLOCKED)

Loose Rock Boulders

Concrete Arch

Method when Sides Sound

CROSS SECTION OF TUNNEL LINING

Heading Approx 9 x 7 x 6

Benching Approx 14 x 4

CROSS SECTION OF TUNNEL AREA 140 SQ FT

6110 Feet
Fall 1 in 261

SHAFT No 4

Fissures Cement Gunned

Line correct Level 6" out Junction 14 5 24

Turn in Gorge

1119 Feet
Fall 1 in 249

1665 Feet
Fall 1 in 250

C L of Tunnel

PLAN

Adit 4

SHAFT No 6

Junction 25 6 24
Line correct
Level correct

Outlet

WEST

Masonry Cement Pointed

15 Dia

SHAFT No 6 (SURGE CHAMBER)

TUNNEL SECTION

Tata Power Co.

and 10 ft. deep and is designed for a maximum head of 70 ft. The sluices are fitted with gun-metal facing working against gun-metal guides on the body. While the gates can be operated by hand, each is fitted with a 440-volt 12 h. p. A. C. motor, which gives a speed of one foot per minute. The gate stem operates in a nut thus ensuring a positive closure. The gates are housed in a steel-framed building, which contains all the electrical and mechanical equipment for their operation.

The gates are protected by screens of 6″ × ¾″ bars, spaced 4″ apart centre to centre.

The bars are built into reinforced concrete beams embedded in the sluice gate piers. The screens are placed diagonally in the approach cut which ensures a natural self-cleaning effect. The net area is 325 sq.ft. for a tunnel section of 140 sq.ft.

The main tunnel is 14,469 ft. long and 140 sq.ft. in section, the connection of the three sluices being by three 8′-6″ tunnels approximately 57 sq.ft. in section and each 100 ft. long with a slope of 1 in 3·8 leading from the sill of the main gates to the bottom of No. 1 shaft. The tunnel has a slope of 1 in 255 and has been designed to convey 1,200 cu. secs. The maximum static head at the bottom of the shaft No. 1 which is R. L. 1898 ft. is 91 ft. and at minimum draw-off level the head drops to 38 ft. The tunnel has 6 shafts and two adits, but driving was never done from all faces at any one time, because of the shorter drives holed through before the deeper shafts were completely equipped. The shaft sinking operations were spread over 18 months. Sinking was begun by hand and machine drilling was adopted as soon as power was available.

In spite of lining, where rock was not met with, the shafts were not water-tight, those on the lake side being particularly troublesome during the monsoon when a rainfall of 6 to 10 inches a day was not unusual.

No. 6 shaft, which is 458 ft. from the out-let and only 295 ft. from the point where the steel bell month is concreted into the tunnel, is designed as a surge chamber to deal with the load of six generators, totalling 1,80,000 h.p. It is 15 ft. in dia. for the lower 100 ft. and 36 ft. in dia. for the upper 59 ft., the crest being at R.L. 2,017 or 28 ft. above high flood level.

The rock met with was generally good Deccan trap and only 1,339 ft. or less than 10% had to be lined, the greater portion of this being near the two adits and at the penstock end. The lining consists of stone masonry (tunnel spoil) in cement mortar for the side walls, while concrete with tunnel track rails is used for

the arch or roof. Several places are treated with cement gun for creaks and fissures.

Principal quantities.—Tunnel

Main approach cut, including deepening length feet	5,000
Excavation (all sorts), including deepening: cubic feet	26,20,000
Three sluice gates (each 8′-6″ × 10′) ...	...
Total weight of set, tons	36
Six shafts (including surge chamber) ...	...
Total length : feet	1,700
Total length lined feet	220
Total excavation : cubic feet	3,10,000
Two adits	...
Total length feet	727
Total excavation cubic feet	54,600
Main and auxiliary tunnels (excluding shafts) length feet	14,850
Excavation cubic feet	20,70,000
Total length lined feet	1,350

With the exception of 168 ft. at the outlet the whole driving was done by pneumatic power. The actual driving was begun in July, 1923, and completed in June, 1925. With an allowance for preliminary work at the bottom of each shaft, the time may be taken as 20 months inclusive of two monsoon periods with an average rainfall of 4″ to 6″ a day continuously for 2 to 3 months. The average daily driving from all faces was 25 ft. and the maximum 45 ft.

The quantity of gelignite used ranged from 8½ lbs. to 13 lbs. per linear foot of 140 sq. ft. of tunnel. All the removal of the spoil was done by hand.

Artificial ventilation was provided only when drives exceeded 1000 ft. A motor-driven pressure blower with a capacity of 2300 to 2600 cu. ft./ins. delivered into a canvas pipe.

Electric lights were fixed 50 to 70 ft. apart, the face being illuminated by a portable cluster of five or six 100 c. p. lamps.

Apart from lighting and ventilation all power below ground was pneumatic, air being supplied at 150 lbs. sq. ins. from six compressors each of 750 cu. ft./ins. capacity and rope driven by 150 h. p. 2200-volt motors. These compressors were placed at the various shafts and were adequate for all drilling, winding and pumping.

Electric power was distributed from a 1500 K. V. A., 22000/2200-volt substation, feeders being run to the various shafts.

Shaft sinking and preliminary work, including plant erection, were done departmentally. The company supplied steel and explosives deducting their costs from the fortnightly bills.

The staff consisted of a tunnel superintendent and two tunnel engineers (one for each side of the main ridge) and a mechanical and an electrical assistant and a medical man.

Andhra Valley

"The tunnel is 8700 ft. long exclusive of cut and cover in valley bed before reaching the entrance to the tunnel.

The tunnel has been blasted out of solid rock throughout and the work carried out by gelignite explosives much depended on the hardness of the rock as regards progress.

To accelerate the progress four separate shafts were sunk over the tunnel line making it possible to drive at 8 points simultaneously. These shafts also provided fresh air to the borers and powder to men below. In some places only 4 ft. per day were worked whilst in other places 12 ft. per day were blasted out.

The hammer jacks in use were operated with compressed air supplied (by air compressors electrically driven).

At the entrance to the tunnel a reinforced concrete structural screen of the parallel bow type is erected to prevent any bulky bodies entering the tunnel.

At a short distance from the tunnel inlet a sluice gate entrance is provided.

The chamber and shaft is of oval section 28 ft. × 19 ft. the major axis in line with the tunnel and extending upwards to the ground level and above the high water level. The sluice gate equipment is in duplicate and can be raised or lowered independently.

A duplicate screen of $1\frac{1}{4}''$ mesh to take up any loose agricultural growths, etc., which have passed through the screen at entrance of the tunnel is installed in the chamber. Either one of the screens can be raised to the top platform for cleaning. The first design of the tunnel was circular in section 10 ft. diameter but later on it was given up and now it is made with sloping sides, arch roofing and flat flooring. The inside is finished with masonry to present a smooth surface to the flow of water, also filling in of any questionable places in the rock.

The tunnel is made straight between the inlet and outlet with the exception of a slight angular direction towards the outlet bringing same to a suitable site to accommodate the pipe lines and headworks.

Adopted from Cursitjs on the Tata Power Company's Hydro-Electric Project excerpt minutes of proceedings of the Institute of Civil Engineers, Vol. 228, session 1928-1929, Part 2.

It has a falling gradient from inlet to outlet of 4% *viz.* 42 ft. throughout including cut and cover. There is a vertical fall of 3½ ft. on entrance to the tunnel which will assist the initial velocity of flow of water. The area of the tunnel is 82 sq. ft.

The rate of flow for 60,000 h. p. is 400 cu. secs. and for 84,000 h. p. 560 cu. secs. Length of the tunnel is 8,700 ft., cut and cover 528 ft., and open cut 300 ft.

A surge chamber 15 ft. dia. is built near the outlet of the tunnel in the rock of the mountain and under static conditions the surge chamber will be full of water to the water level of the lake—2,195 ft. H. W. L.

The function of the surge chamber is to relieve any undue stresses that may arise in the construction work and plant due to cutting off the supply by the penstock valves or stoppage of the water from any other cause arresting the momentum of the water. Under such circumstances, the water would rise in the surge chamber above its original level and balance the water momentum; to close all six units would mean a rise of 46 ft. and of one unit only 6·6 ft. based on the standard formula $Kwv^2/2g$ as the water momentum and its equivalent water height, the constant K being taken as ·6. However, the valves operating in series have a time setting of 150 seconds when closing.

Tunnel, Mahora

Tunnels may be of pressure type or non-pressure type. When of considerable length, they are of the pressure type, so that the drops may be utilized as usual head. Tunnels are safer and their upkeep is usually low as compared with the open channels, especially if these are built on the hill-side where they are exposed to dangers from boulders striking them, under mining, etc.

There are six tunnels for the Mahora flume in the length of the flume line at Mahora. Through the first five of these the water is carried in the wooden flume itself and only in the last one the water goes through the tunnel itself. This one is lined with bricks and mortar and not with concrete.

Length of the	1st	tunnel is	... ...	90 ft.
,,	2nd	,,	... ...	400 ft.
,,	3rd	,,	... ...	450 ft.
,,	4th	,,	... ...	250 ft.
,,	5th	,,	... ...	250 ft.
,,	6th	,,	... ...	553 ft.

Health on Andhra Valley, pp. 30-32, The Institution of Engineers, Vol. III, April, 1923.

Tunnel, Mandi

The tunnel has been primarily designed for a discharge of 600 cusecs. at 9 ft./sec. The same tunnel will be capable of meeting the peak discharges of 900 cusecs (anticipated as likely at future date) at 13·5 ft./sec. Any increase in this value will necessitate a lowering of surge-shaft levels and increase the head of the tunnel at its lower end which is already 260 ft. when the dam is full.

Size of the tunnel for 900 cusecs. and 13·5 ft./secs. is :—

$$\frac{\pi D^2}{4} \times 13{\cdot}5 = 900$$

$$\therefore D = 9{\cdot}21 \text{ ft. or } 9'\text{-}3''.$$

But velocity 9 ft./sec. or $13\frac{1}{2}$ ft./sec. will be impossible unless the tunnel is lined.

The following table gives an idea of the cost of lined and unlined tunnel :—

No.	Date.		Lined	Unlined.	Remarks
1	Friction coefficient	...	·014	·027	
2	Discharge cusecs ...	a ...	600	600	
		b ...	900	900	
3	Velocity	a ..	9′/sec.	4·11′/sec.	
		b ...	$13\frac{1}{2}$′/sec.	6 16′/sec.	
4	Friction losses °/o feet	a ...	2·23′	2·23′	
		b ...	4·77′	4·77′	
5	Diameter ...	...	9·215′	13·6′	
6	Cost of concrete per ft. run	...	Rs. 8·2		
7	Amount of extra excavation per ft. run at Rs. 60 °/o.		...	Rs. 45 6	

Only that portion of the tunnel which passes through Gneiss where the overburden exceeds 1,000 ft. is compared in the table. The lower end of the tunnel which is under pressure in schists must be lined. So it is better that the whole of the tunnel is lined.

The tunnel is 14,200 ft. long within internal diameter of 9′-3″ lined with concrete throughout so as to provide reliable friction condition and at the same time to ensure its being water-tight

under the comparatively high hydro-static head to which it shall be subjected after the completion of the dam in the second stage.

The tunnel has two exits, one for scouring purposes only through the constructional adit approximately 1,200 ft. long which has been driven from the Wayer Nala from R. L. 5,760, so as to reduce the total drive through the mountain range to 11,100; the second and the main exit into the surge-shaft where the tunnel proper ends. It then bifurcates into two 6 ft. diameter steel pipes each concreted up solid in one of two separate pipe tunnels, each 1,200 ft. long emerging at the surface at R. L. 5,635 ft.

Blow-off Valves:—Silt and sand deposited in deep depression in reservoirs should be carefully investigated and are removed through blow-off valves located in the bottom of the conduits at the lowest point of the profile. If the rate of rise is gradual and the velocity high, the deposits will be negligible.

Silt Basins

Silt Basins:—When the intake for the canal is at a diversion dam in a rapidly flowing river, the velocity of the water in the canal may, at times, be very much less than that in the river, and under such conditions deposits in the canal are likely to occur. Even in cases where the velocity in the canal is high enough to keep the sand in suspension, the resulting wear on the sealing rings of the turbine may be a serious matter, increasing clearances and decreasing efficiencies. To obviate this difficulty, the use of sand boxes and settling basins have been introduced. The problem is much the same as that presented in the design of settling basins and cogulation basins in filtration work, except that the permissible velocity is much higher. When there is a terminal pond or a large forebay, the velocity through them is so low that sand of a size to injure the turbines is deposited. If the forebay is utilized for this purpose, special means for removing the sand, akin to those described below, are frequently advisable.

Position of silt basin:—The best position for this device is at or near the headworks though another one near the forebay may also be indicated. If the settling tanks allow the velocity of water to be reduced to 6 in. per sec. or less, most harmful matter will deposit very soon. The water will enter and leave the tank with practically no change of level and the entry and exit will be tapered and graduated so as to reduce eddies and disturbances. There should be considerable depth of water in the tank below the level of the bottom of the channel and the

bottom should slope steeply towards the scour gates. Preferably there should be several compartments, each with its own gate, each should be separated from the next only by a low barrier consisting of the higher level bottom of the adjoining compartment. Thus, when the scour gates are opened, with the channel running full, water will pour into the tank with increasing velocity from both entrance and (for a time) exit, and much of the deposit will be washed away. The rest will need very little handling to follow suit, and the interruption to supply will not last long. If the regulating storage lower down is too precarious to allow of this emptying of a considerable length of channel, a gate can be placed at the exit of the silt tank to prevent any back flow. So long as there is an excess of water, a small proportion can be allowed to escape continuously under the scour gates, in which case it will carry with it all the matter that is kept rolling along the bottom.

Rate of deposit of silt :—The rate at which muck, silt, etc., is deposited in a reservoir depends upon the quantity carried by the stream, the size of the reservoir, the volume of the stream flow, and the character of the catchment area. If the capacity of the reservoir is small compared with the average annual discharge of the stream, it will act in the nature of a retention basin, and when the stream is in flow and carries its greatest load of muck or silt, a portion of the muck or silt goes over the dam. On the other hand, if the reservoir capacity is large compared with the annual stream flow, little goes over the dam, and the muck and the silt brought down by the stream is deposited into the reservoir—*vide* pp. 214-215.

There is probability of deposits of silt destroying the effect of both pondage and storage.

Removal of Silt

For sand and gravel the eroding and transplanting are not very different but for clays and silt the difference is very great. Thus when a plant is shut down for sometime and silt, clay, sand and gravel are deposited, but where the plant is restarted after some time the sand and gravel are picked up and transported with the water but silt and clay are not so removed.

A velocity of from 2 to 3 ft. per second will be sufficient to prevent the deposit of silt.

*Kennedy formula for this is $v = cd^{0.64}$

Where c = a coefficient which varies with the character of silt between 0·82 and 1·07.

*Remodelling and Distributaries on Old Canals Punjab Irrigation branch papers No. 10—J. S. Beresford and R. G. Kennedy & T. Highhame.

V = mean velocity in canal which will prevent the deposit of silt. d = depth of water in feet.

Silt Reservoir :—The water in the river Cauvery carries a considerable amount of silt in suspension and it is found that large quantities of it are deposited in the forebay. This necessitates the employment continuously of a new sowelis to remove it. Unless the silt is taken out regularly, the final result would be the complete shutting down of the station. So this forms a very serious question. With this object in view, a large silt reservoir has been built and its location is at a point in the channel about a mile from the Bluff, where it forms a large horse-shoe bend, entering and exiting in the same line of horizontal inclination to the silt tank. The reservoir is constructed by putting up a dam across the horse-shoe and breaking the channel, so that a large lake is formed. Since the course of the water is completely changed through 180°, the water loses its velocity on entering this reservoir, due to the change in the direction of flow of water, the greater part of the silt will be deposited here in this reservoir. It is calculated that it will take about 40 years to fill up the reservoir basin with silt. Thus this silt reservoir relieves the forebay of deposit. With silt-reservoir, time taken by the channel water to flow from the headworks to the forebay is 2·55 hours. Without this reservoir, it is 1·75 hours. There are two scouring sluices at silt reservoir with two vents of 6′ × 6′ each. The capacity of the reservoir is 5 million cu. feet with a catchment area of 1·8 sq. miles.

Fig. 51.



Cleaning the Silt.

At the time of cleaning, the silt basin is provided an outlet to the river through a gate and a channel. Together with the water, all the silt basin is sent to the river.

The operator, incharge of the headworks has also to look after the silt basin, but

when cleaning takes place, extra coolies are sent from the power-house.

Silt Basin at Mahora

About $\frac{1}{2}$ mile from the headworks the flume water is led into a shallow tank, known as the silt basin, where the velocity of water is reduced to 2 ft. per second, owing to the expanded volume of flow and the silt and mud carried along with it from the river are deposited here. The tank is 600 ft. long and 83 ft. wide and during one year the level of the bottom of the silt basin rises by about 3″ or 4″ inches due to the deposition of mud. There are sluice gates at the sides of the basin so that the tank may be cleaned periodically. About 150,000 cu. ft. of silt is removed every year from the silt basin.

(*b*) **Silt Basin** :—For the drawing of the silt basin—*vide* Fig. 52.

It forms an integral part of the headworks and hence it is just near the headworks. There may be more than one silt basin and at different places in the flume, if some rivulets are joining the flume.

The silt basin is a big rectangular tank with round corners and about 200 yards by 30 yards. The flume enters the tank on one side and proceeds further on from the other side. The function of the silt basin is quite clear.

It is found very essential that the water going to the Pelton wheels should have no silt in it. There should be no granules of sand in it, as even the smallest granules are harmful at high velocities. These granules rub off the material of the buckets of the Pelton wheels. The worn wheels require replacement very soon, and the efficiency is also greatly reduced. The silt basin is quite an important part of the hydro-electric installation. The tank is cleaned once or twice a year, and during the cleaning, some separate path must be provided for the flume water to go, otherwise shut-downs must occur.

The arrangement is shown quite clearly in the Fig. 52. The auxiliary path allows the flume to go, as it is, *i.e.*, without passing through any silt basin. The necessary control arrangements, by means of gates, are provided.

When the silt basin is to be cleaned, the two flume gates at the entrance and exit of the flume to the silt basin are closed and the sideway gates are opened. This allows the water to go by the auxiliary path, which meets the flume line proper, at a certain distance beyond the silt basin.

Fig 52.

Silt basin at Mahora.

Electrical dredging on the Jhelum :—This has been successfully used to clean the silt and debris from the river Jhelum. The dredging out of the bottom of the river Jhelum for a few miles of its length in the Srinagar valley was first proposed by Major DeLotbiniere, who was responsible for the designing of the present hydro-electric power-house at Mahora. The vast quantity of accumulated silt on the bottom of the river had raised the bottom to such an extent that the river would spread out every year in summer throughout the valley when the snows begin to melt. This water used to inundate the whole countryside, just where the lands are very fertile and the town of Srinagar also used to be flooded in some parts. During the designing of the Mahora power-house, it was pointed out by the said Major that if the bottom was cleared of the deposited silt just where the river came out of the Wollar Lake, the water accumulated would flow out more readily than before and the risk of the floods could be removed thereby, and at the same time he proposed to do the work electrically, so that the project could be carried out very economically, the power being obtained from hydro-electric power-house. The electrical dredges were the first of their kind in India, if not in Asia, manual or steam power being absolutely unnecessary in their operation. The work was carried for a few years and has now been stopped a year back as it was not found necessary to keep it going continually.

Fig. 53.



A view of the dredger Kashmir on the Jhelum at Dobagh.

The dredging fleet consisted of one dipper dredge, two hydraulic or suction dredges, two clam-shell derries and a floating substation to receive the power from the overhead transmission lines and distribute the same to the various dredges. 30,000 volts are received and 2300 volts lines are sent out from the substation to the motors on the dredge.

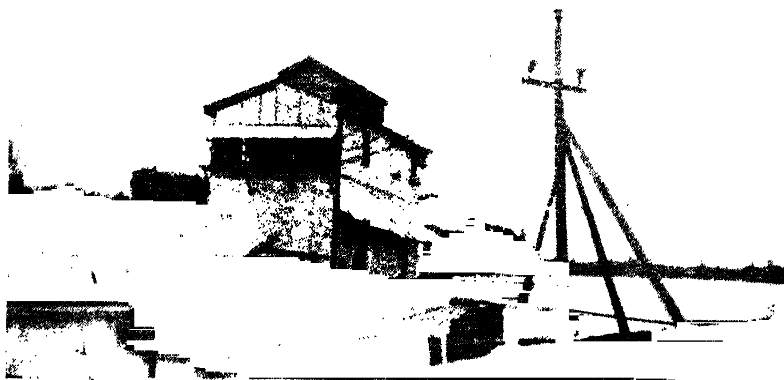
Fig. 54.



Removing the dredger.

The floating substation is towed to the spot of the work and is kept anchored at the spot. Temporary lines run from the overhead transmission lines near-by to the substation. In the substation cabin there are three 200 kW. single-phase transformers connected delta-delta and of the two 3-phase 2,300-volt lines,

Fig. 55.



Floating substation.

one goes to the dredges and the other to the neighbouring big town of Sopore about 6 miles away from the present spot.

The main dredge consists mainly of a cutter, a suction pump and a "stern spudd machinery" meant for advancing step by step. The cutter is at the end of a long shaft about 30 ft. long and 8 inches in diameter. The shaft is fixed to a bevel gear of 8 ft. diameter, which is driven by another bevel gear of 2 ft. diameter. The shafts of the two gears are at right angles to each other and that of the second gear is driven directly by an electric motor of 150 H. P. placed on the deck of the dredge. The speed of the cutter, when in operation, is about 10 r. p. m. Because of the bevel gear arrangement of cutter, shaft may be inclined downwards or hoisted up without disengaging it from the cutter motor.

The cutter proper is of cast iron and has four blades on it fixed on a ring of 6′ diameter at the back. This ring is rigidly fixed to the rotating shaft. The blades are fixed on the ring inclined at an angle of about 50°. The cutting edges are of steel and can be detached for resharpening or repairs by taking out the 10 rivets which fix each to the corresponding blade.

The width of the dredge hold is about 20 ft. and its length is about 60 ft. Throughout its entire length runs a pipe of 6 inches diameter and is connected with the centrifugal pump placed in the middle of the hold. The pipe extends to the front to the cutter and is hoisted up or down along with it when necessary, there being a flexible hose coupling near the hinge. At the back, the pipe extends quite a long way and is carried over the water to the bank of the river on two pontoon barges. The total length of the pipe is 580 ft.

The main centrifugal pump is coupled directly to a 350 h. p. Induction motor placed alongside inside the hold. The capacity of the pump is 80,000 gallons of pure water per minute. When a mixture of mud and water, coming from the bottom of the river, consists of about 70% of mud and 30% of water, the rated capacity is 4 cu. yards per second or 240 cu. yards per minute. On actual runs the capacity has been found to be much in excess of this value and as much as 400 cu. yards have been found to be dredged out in one minute.

The pump-driving induction motor is, by General Electric Co., as are all the other motors installed on board, and has 3-phase, 25-cycle supply, the current taken being 85 amperes at full-load. Its over-all diameter is about 6 ft. It is kept cooled, when in operation, by means of a separately driven fan, which is run by a motor of 1 h. p., supplied with 220-volt, 3-phase, A. C.,

from two small open-delta connected transformers placed on the deck. The lamps for illumination at night are also supplied from these transformers. The capacity of each is 5 kW. The work of dredging was used to be carried out day and night.

Inside the deck-cabin, just behind the cutter motor, are three drums placed side by side. The central one is for hoisting the cutter by means of wire-ropes and pulleys. The ends of the wire-ropes wound on the outer drums are firmly anchored on the banks of the river, one on each. Both these wire-ropes are kept tight when the dredging is going on and one of them is slowly wound on, while the other is unwound. This makes the dredge move across the river from one bank to the other and this is done when transverse dredging is going on. The three drums are rotated by the same motor through bevel gears slipped in or out of engagement by the movement of control levers in the operator's cabin. The rating of this Bow-winch motor, as it is called, is 50 h. p. 2,080-volt, 3-phase, 750 r. p. m. This is also by the General Electric Co.

At the back of dredge are two vertical logs of wood, one $97' \times 3' \times 3'$ and the other 90′ long and $2\frac{1}{2}'$ square, placed one behind the other on the central line of the dredge. They are placed 4 ft. apart. These vertical posts go down to the bottom out of the river and get struck in the mud, their ends being pointed. Each of them can be lifted up separately at will by an induction motor of 30 h. p. run off the 2080-volt supply. This arrangement is called "*Stern Spudd Machinery.*" By means of this arrangement the dredge can be made to advance step by step or can be made to rotate about any one of the posts. To advance a step, the front post is kept stuck up in the mud and the back one is lifted up clear of it. By working the Bow-winch motor the dredge is then rotated about the front post and then the front post is lifted up after the back one has been sent down. By a repetition for the rotation in the other direction a step of about 4 ft. is advanced.

To pump out any water that may leak into the hold, two centrifugal pumps are placed in the hold. A 10 h. p. induction motor drives these pumps and gets its current from the 220-volt, 3-phase supply.

Control room.—The operator sits in the control cabin on the roof of the dredge and all the movements of the dredge can be cotrolled by him. The starting compensators for various motors are all placed near his reach. At the back of the room is the switch-board for housing the ammeters, voltmeter and oil-switches. On both sides of operator are 5 levers for changing

the gears in the Bow-winch machinery and for applying brakes on the various motors. In front is placed a vacuum gauge for the inspection of the suction in the main pipe.

Operation :—As the cutter rotates in the mud at the bottom of the river, the dislodged mud is sucked up by the suction pipe and is thrown to the bank by the action of the same centrifugal pump. It takes only 5 seconds for the mud to travel through the entire length of 580 ft. of the pipe. By varying the inclination of the cutter shaft various depths can be reached below the water, the maximum being 26 ft. The mud can be thrown up on the banks to the maximum height of 40 ft. above the water level.

Taking 110 lbs. as the weight of one cubic foot of the mixture of 70% mud and 30% water, and taking 300 cubic yards as the quantity sent up in one minute as the average during the day and taking 30 ft. to be the height to which the mud is sent up, we get the h. p. developed by the motor in operation, neglecting all losses due to friction, etc.

$$= \frac{110 \times 27 \times 300 \times (26+30)}{60 \times 550 \times 5} = 300 \text{ h. p.}$$

as the time taken by the mud to reach the top is 5 seconds.

When the dredge is working at its best, the h. p. developed by the pump motor is $300 \times \frac{4}{3} \times \frac{66}{56}$, *i.e.*, 479 h. p. When the cutter motor, Bow-winch motor, the suction pump motor and the stern spudd motor are all simultaneously working, the dredge is taking the maximum power, which is almost equal to 1000 kW.

The dredges were worked day and night and the maximum length of the river bottom that could be dredged out per day was near about 200 ft., and where the bottom was found to be exceptionally hard, about 30 ft. to 50 ft. could be advanced per day. The average depth to which the bottom scoured was about $1\frac{1}{2}$ ft. The width of the river where the dredging was performed is nearly constant and is about 400 ft.

The *immediate result* of dredging was to save the whole country-side from the annual devastating floods. By providing an easy outlet to the accumulated water a vast area of cultivating land was reclaimed which was about 50,000 acres in area. The original idea was not to dispense with the dredging fleet altogether, at any time, but about a year ago the whole concern was shut down.

The total cost of the dredging fleet came up to about Rs. 10,00,000, *i. e.*, 10 lacs of rupees. The interest on this sum and the depreciation at $4\frac{1}{2}\%$ came to about Rs. 90,000, which sum included the repair charges. The annual establishment charges were about Rs. 48,000. The cost of electric power consumed per year at Mahora was Rs. 1,80,000 ; so the total expenses per year used to come to about Rs. 3,18,000.

The reclaimed land yielded a revenue of Rs. 4 per acre or a total of Rs. 2,00,000 per year. The remissions in the revenue which had to be granted previously because of the annual floods used to come to about Rs. 1,00,000. This amount was thus indirectly saved as a result of the dredging work as well as the money spent every year in the relief work in the flood-stricken area. Taking all these into account, the annual income was about five lacs of rupees as the result of the dredging, or a net gain of about two lacs of rupees to the state, subtracting the 3 lacs of annual expenses for maintaining the fleet in the working order.

The Forebay :—It is an enlargement from the conduit at its terminal for the purpose of lowering the flood velocity and distributing it to the turbine chambers of the power station. Its function is that at all times the varying demand of water will be met and pipes taken off from it will invariably be fully submerged. Another function is to prevent any foreign matter entering the pipes. Thus there are revolving fine screens and strainers in the forebay and there are gates and valves by which any exit can be closed.

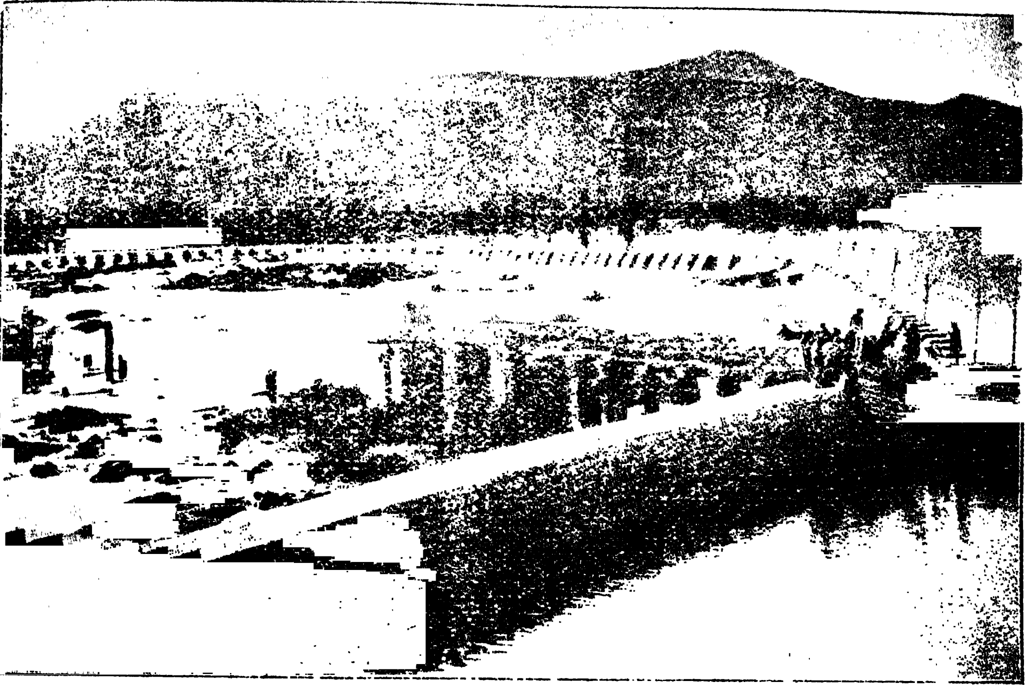
Fig. 56.



Central View, Forebay.

It may be the pond formed by the **diversion dam,** *vide* Fig. 57, or it may be the enlarged section of a canal which is spread out to accommodate the required width of the intake and will be of sufficient capacity to meet the varying demand as the load alters unless there is other storage for the purpose and not only can it be drawn upon when the demand suddenly rises, but it can take up or get rid of the unwanted supply coming in if a shut down occurs.

Fig 57.



Diversion dam, 2,200 ft. long across the Cauvery river above the falls.

This may be intended mainly (1) for regulation, or (2) holding considerable reserve supply. It must be so designed that, while acting as an efficient surge tank, and as a regulator to deal with loads temporarily exceeding the capacity of the channel, it does not allow any foreign matter to get into the supply pipes. Coarse and fine strainers should be provided, and they should be of such breadth that the full flow required by the wheels can pass through them if the water accidentally falls low. This is often overlooked in designing. In some small installations the actual reservoir is at a little distance from the forebay or pentrough, and the two are connected by a closed pipe large enough to deal with the peak load. In such cases the reservoir performs these latter functions, while the forebay proper becomes only a surge tank.

Fig. 58



Forebay end of Power Channel, earth between two sides is visible.

The forebay, or forebay reservoir where these are combined, should, if possible, be sectionalized or duplicated to admit of cleaning out; and the design should be such as to facilitate this, as suggested above for silt traps where the channel discharges into it, which should be at a point as far as possible from the pipe entry, there should first be a small chamber to take eddies and induce still water; this will also catch some of the residual silt, and as its walls will stop a foot or two below normal water-level only the clean overflow will pass into the reservoir proper, and not the bottom layers, except at cleaning-out times. (This, of course, assumes that the width of the sill is greatly in excess of the width of the channel). The spill-way and scour valves of the reservoir should be near this point, where most of the mud will collect. The strainer, that finally ensures no foreign matter getting into the pipe, requires careful thought. The obvious plan is to put it immediately behind the bell-mouths of the pipe, and to close it in at the top. But there is a chance then that some part of it may eventually rust away and get into nozzle. Repairs would be difficult without emptying the forebay and shutting down the plant, and it has been found in practice that the arrangement is not satisfactory. The alternative design, which should be adopted, is to put the strainers in the main forebay, at the entrance of chamber feeding the pipe, and this chamber should be covered over. Rolling screens, operated by a winch, are useful because of the facilities they offer for cleaning, but the wear and tear of this type is great. A point of importance is that the screens must be capable of passing the full flow when the forebay is nearly empty. The racks in the forebay should be of finer pitch than those at the intake, in order to catch any small debris passing the intake screen and any leaves, etc., which may

enter the head race between the intake and the forebay. The pitch should not exceed about 1 inch.

An emergency by-pass from the channel, also provided with a screen, should be made to discharge direct into the bell-mouth chamber. If repairs are required to the reservoir, this will prove invaluable, and when the water is running clear, it can be used while cleaning work proceeds.

The capacity of the forebay should be such as to enable economy to be practised without risking, on the one hand waste, or on the other hand, shut down due to an empty forebay. An hour's full-load supply in hand with the possibility of saving a further half hour's supply or so in case of a shut down, is about the minimum as to give security and to avoid waste. Sometime the ground will not allow so large a forebay ; but where the site is favourable, considerably more storage is advisable. If there is no other storage on the system, a much larger forebay than the above is necessary. If the load factor is bad, the forebay might store water which may be used at times of heavy demand and thereby increase the capacity of the plant three or four times.

Fig. 59.



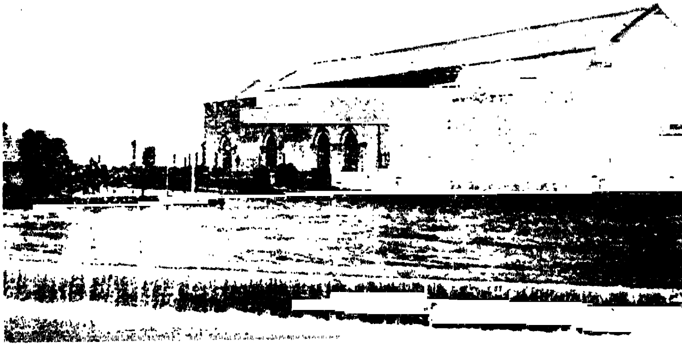
Forebay from a distance.

Forebay (Cauvery)

The forebay at the tail end of the channels flanked by the lofty and imposing transformer station, as shown in the Fig. 60, is situated on the brow of the bluff about half a mile below the western or "Gagan Chukki" falls and opposite the Northern end of the island of Sivasamudram.

The forebay originally built had to be enlarged in 1926 owing to the increased load demand.

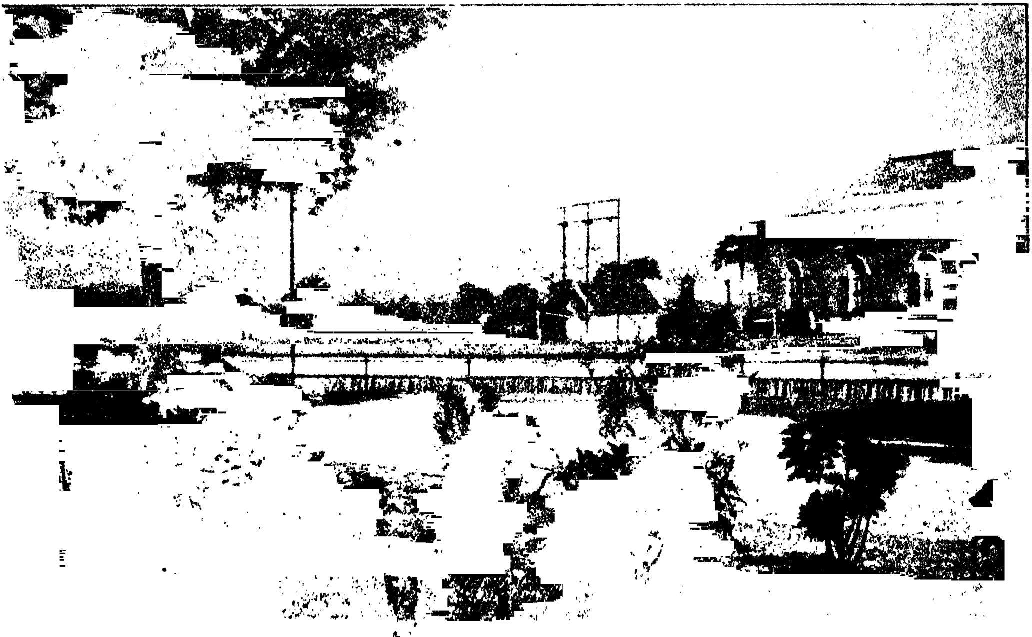
Fig. 60.



Forebay, Cauvery.

A view of the forebay and waste weir is shown in Fig. 61. The

Fig. 61.



Forebay and waste weir.

Fig. 61 (*a*)

Underground Channel.

11'

10'

18'

V

S

S

Floor of Cabin

G

Sill

P

T

P

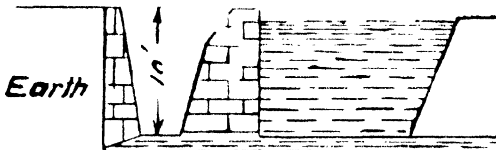
3

A

$10\frac{1}{2}$'

4'

S. S.—Eight Sluice gates.
T—Telephone Cabin.
G—Grating of iron.
P.P.—Pipe Lines.
V—Valve for opening a gate for washing.



Section at middle

FOREBAY AT MAHORA

To face page 277

entrance to each penstock chamber is protected by an iron rack which prevents the entrance of debris into the penstock. Each penstock is also fitted with a form of automatic valve which comes in operation to close the penstock, when the velocity of the flow increases a predetermined value owing to any contingency. There are three different types of valves in use in the forebay, which are entirely mechanical in operation, but which can be tripped from a distance and the third is operated by an electric motor. The motor is controlled by a switch operated mechanically by a paddle projecting into the water flowing in the penstock. An escape weir 100 ft. long on the left flank takes care of all the surplus water reaching the forebay, and the only *defect in the forebay* is that there is no alternative path for the water when the forebay has to be cleaned. That is why there is a general shut-down once in a year for six hours when the forebay will be cleaned. This will be a spectacular sight indeed and the cleaning of the forebay is shown in photo in pages 232-233.

Forebay, Mahora :—The forebay is just above the power-house site at Mahora and it may be described as a connecting link between the flume and the pressure pipes, leading to the Pelton wheels. The plan of the forebay is shown in Fig. 62 and some important dimensions have been marked. Its height is exactly 396 ft. above the floor of the power-house. Just at this place the hill is quite flat and the tank of the forebay has been excavated just at the edge of the hill.

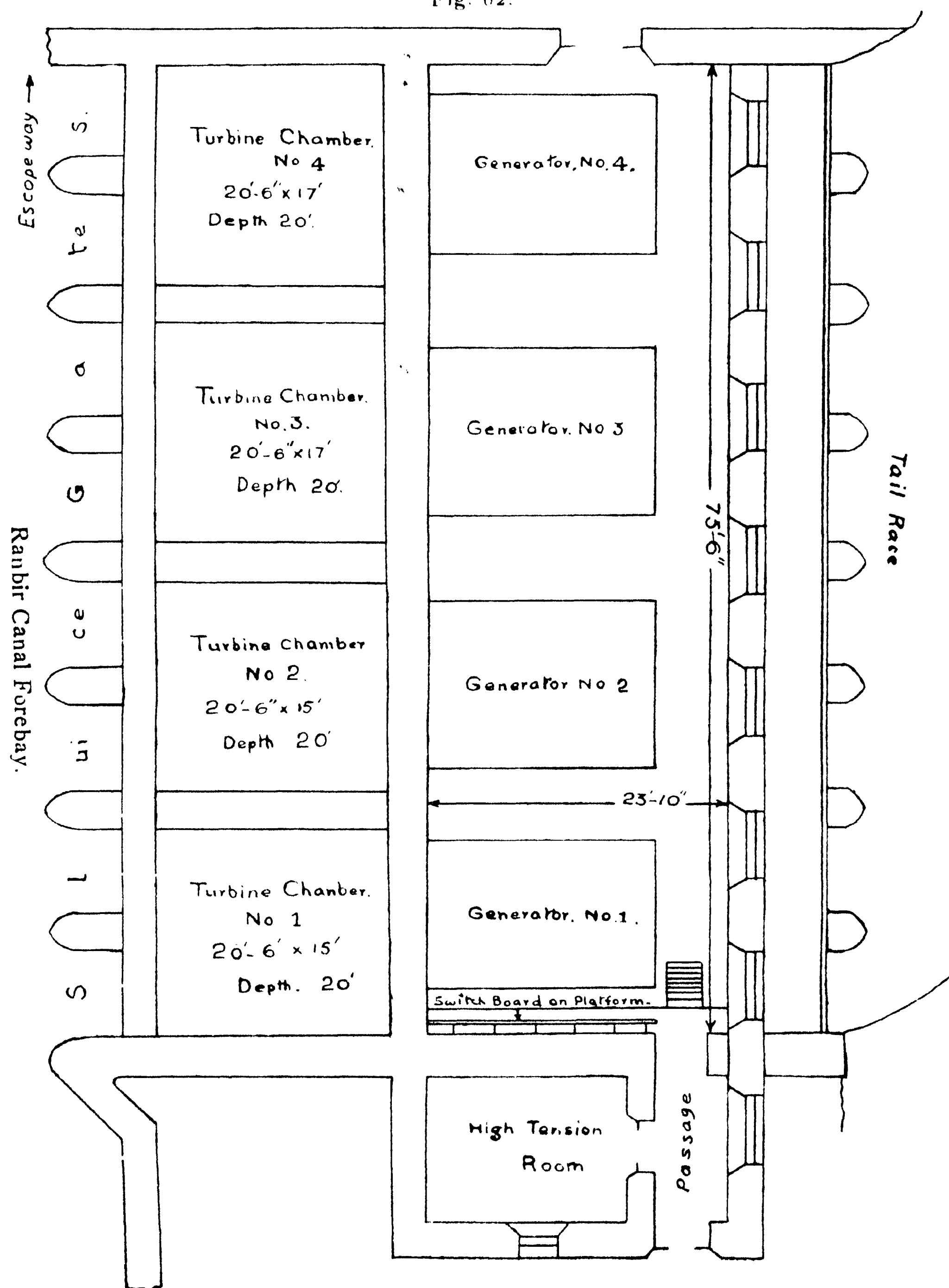
The size of the flume line near the forebay increases slightly to about 8′ by 10′. This has been done to slightly reduce the velocity of water. Just at the entrance of the flume line to the *forebay tank* is a *fine strainer* going about 5 feet deep in the water. The depth of the water here is 8 ft. This strainer checks all floating material.

The forebay is about 20 ft. wide and 50 ft. long. The water depth in the forebay is not the same everywhere. In the first half portion of the forebay the depth is 8 ft., while in the other half the depth is 14 ft. The object of making this change in the depth is to allow silt to accumulate in the deeper part from where it is removed occasionally.

There are 8 gates in the forebay on the power-house side. The first four gates supply water to the pressure-pipes, while other four are for extension purposes. The gates are of timber and work on worm wheels operated by hand.

Before passing through the gates, the water passes through a strainer, in order to ensure that no solid material goes to the pipes.

Fig. 62.

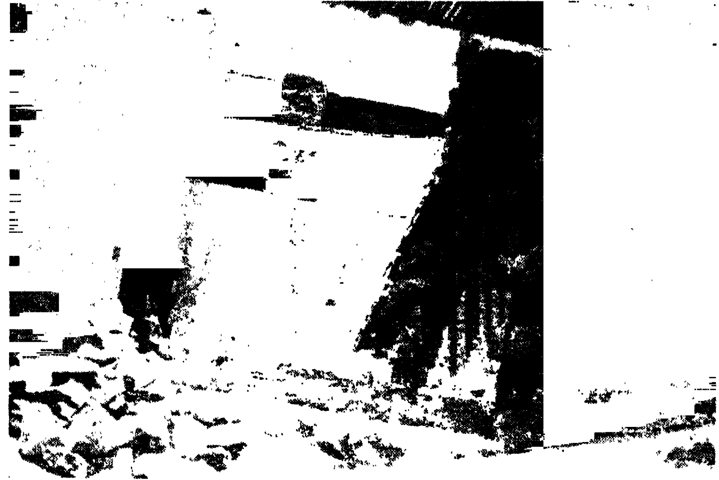


Plan of Power-house Jammu,

The floor of the the tank is of brickwork steps-down in two steps towards an underground channel opened by a sluice gate so that the floor can be cleaned, if desired.

The sluice gates are placed inside a cabin and there is telephone connection between the power-house and the forebay as well as with the headworks to facilitate in the regulating of the flow of water into the forebay.

Fig. 63.



Construction of sluice gates, Cauvery.

The ends of the penstocks are burried in the brickwork foundation of the forebay.

There is a *trolley line* on the sloping hill-side, running parallel to the pipe lines and the wire connecting the two trolleys on the two parallel tracks is wound over a drum of about 6 ft. diameter, so that by rotating the drum the trolleys can be raised or lowered. When one of the trolleys is being raised, the other is going down. This is for facilating the inspection of the pipe-lines—*vide* Fig. 65.

Fig 64.



Surplus Sluices, Mettur.

By the side of the forebay and separated from it by a wall is the overflow arrangement (channel), which is continuously taking in the surplus water and discharging it into the Mahora Nala. In the forebay itself, there is a rectangular notch about 14 ft. above

the bottom of the forebay. When the water level reaches the notch, the water is discharged into the overflow channel from where it goes to the river through Mahora Nala.

Fig. 65.



Trolley Track General Station.

A gate has been provided at the end of the forebay for the purposes of cleaning and removing silt. Through this gate, the silt is made to go with water to the Mahora Nala, when the forebay is required to be cleaned. It is very seldom that the forebay is cleaned, as the water coming from the flume is sufficiently clear.

The **defect** is that, when the forebay is to be cleaned, the power-house has to be closed, as the supply pipes cannot get any water. The designers ought to have provided a by-pass to the reservoirs direct from the flume. In that case, the gates could have been closed and the forebay cleaned without stopping the power-house machinery. If at any time the forebay requires repairs, a bye-pass will prove very valuable. This defect of the design can be remedied even now. These small points of design may spell the difference between success and partial failure, though of course, a good deal depends upon, whether clean or dirty water is generally brought in.

All the work at forebay is carried on by shifts by one man known as the hydraulic attendant. His duty is to open or close the gates as asked from the power-house by telephone. Usually, the gates are not interfered with, as work can be carried out with the help of valves in the power-house.

Forebay, Tata

The forebay has a storage capacity for 1½ hours with all the 8 sets working.

Some difficulty was experienced in finding a suitable material for the dam, the main part was carried down to solid rock and the sides into hard earth. In order to prevent leakage through the wings the whole bottom of the forebay is covered with concrete and faced with a material known as 'Mathoid.'

Openings are provided in the dam for three 82″ pipes. The inlet openings are 8′ 6″ × 8′ 9″ and the gates are arranged to be 'drop-tight.' This is accomplished by bronze-facing strips at the sides and by rubber-packing at the top and bottom. The gates are made of sectional iron plates and angle pieces connected together by rivetted joints. The gates have six rollers on each side. These rollers have to bear the pressure of the water when the gates are lifted or lowered, and permit of the gates closing automatically by their own weight.

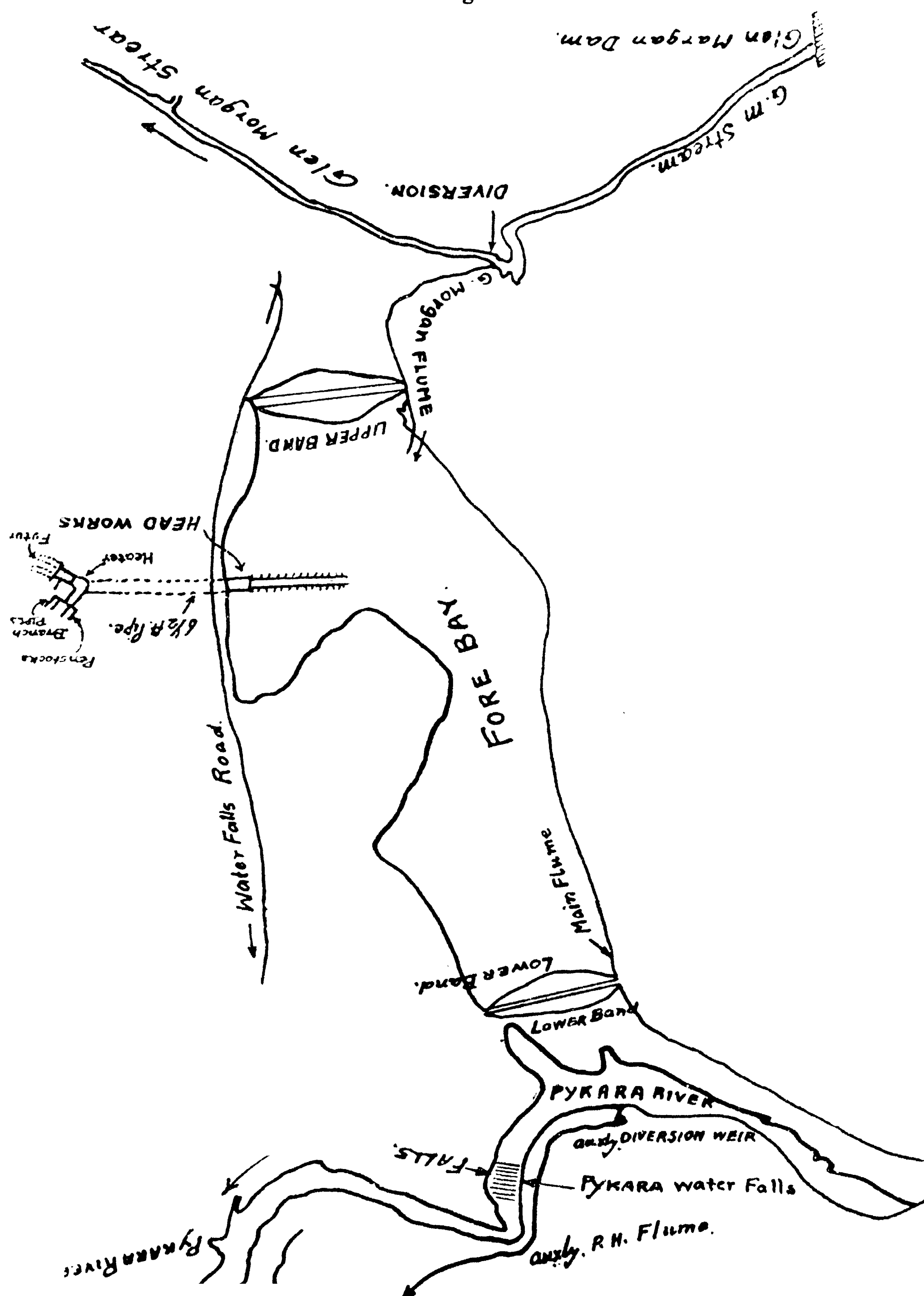
The gates are opened by rocks either by hand or by an electric motor. Connected to each gate is a pinion to which is attached a single worm wheel. The worm wheel is connected to the shaft by means of hand engaging and disengaging couplings. The motor is of 12 h. p. 600 R. P. M. This gives to the gates a lifting speed of 13½″ per minute. So that to open fully the gates take about 8 minutes.

Special provision has been made for speedily closing the gates from the power-house in case of a burst pipe or any other necessity. This is accomplished by a solenoid releasing the ratchet wheels so that the gates close by their own weight, the weight being such as to overcome water pressure. On the gates there is a device which automatically disconnects the electric motor before the gates reach the highest or lowest positions. The gates are also fitted with automatic brakes, so that when operating the gates from the power-house they cannot be damaged by filling too quickly.

The inlets are protected by screens so constructed that one screen slides over the other, the object being to free the screens from grass or other debris collected there. Sludge discharge gates are provided in the forebay and every means is taken to prevent unsuitable matter getting into the down take pipe.

Forebay, Pykara :—As has been pointed out, every hydro-electric project requires a lake or water-shed at the beginning of the pressure line or penstock to regulate the flow in flume due to constantly varying load conditions. A certain amount of pondage is necessary also for emergencies. In the Pykara scheme, the forebay is a reservoir too. So it performs the functions of a forebay beautifully well. It has an effective capacity of over 50 million cu. ft. which is equivalent to about 2675000 k.W.H. There is a natural depression of the country on the summit of the main hill slope between the cliff and the tea estates and this natural valley is being dammed at the two ends by earthen bunds to form a lake. The difference in elevation of the ground between the

Fig. 66.


Glen Morgan Dam.
G.M Stream.
Glen Morgan Stream
DIVERSION.
G. Morgan FLUME
UPPER BAND.
HEAD WORKS
Futur
Heater
Branch Pipes
Penstocks
6½ R. Pipe.
FORE BAY
Water Falls Road.
Main Flume
Lower Band.
Lower Band
PYKARA RIVER
auxly. DIVERSION WEIR
FALLS.
PYKARA water Falls
auxly. R.H. Flume.
Pykara River

two bunds is of about 20 ft. and so the lower bund has to be designed for a greater pressure. The upper bund is about 40 ft. high and the lower bund about 62 ft. high at the deepest point. It will be seen from the plan that the forebay is fed from Pykara at the lower bund and from Glen Morgan dam at the upper end. The elevation at full water level of the forebay is fixed to be at about 6470 ft. M.S.L.

Fig. 67.



The Forebay, Pykara.

The ground for the foundations of the inlet tower and the dams was well explored by test pits and borings and the choice of the fixed capacity and height was found safe. The ground was found to be swampy with a shallow layer of peaty soil followed by layers of sand clay and bed rock. Seepage is expected in places, for instance, at the inlet of the lower end ; cut-off walls are now being constructed to overcome that.

The upper bund is a simple earthen dam. No spill-ways are to be provided and no special precaution is contemplated for the final closure of the dam.

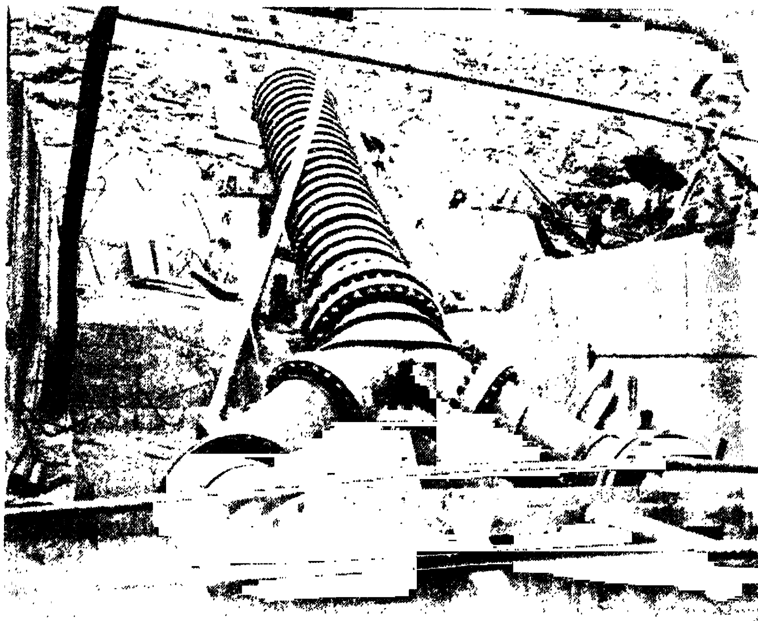
Th lower bund has been designed, of course, differently to withstand the greater pressure. It is too high a dam (62 ft.) for an earthen bund. A concrete core wall, down to permanent bed rock, is designed and the construction of it is now progressing. A core wall is used to ensure a water-tight structure at this head to prevent leake, and to facilitate the construction of the outlet works. Animals will also be prevented from borrowing through the banks. It may be necessary to drain the lower part of the forebay, though, of course, at rare intervals, and a pipe 18″ dia. is buried through the lower bund. The drain water joins the river. A simple flap valve will be installed on the forebay side of the pipe and will normally remain open. A gate or needle valve for regulating the drain off is to be fixed at the other end of the pipe at the toe of the dam as seen in the plan. Should this valve require repairing—and it is only reasonable to presuppose it—the flap valve can be used to close the pipe.

The area of the forebay is about $\frac{1}{3}$ sq. mile and maximum flood flow to be expected is over 600 cusecs. A spill-way is going to be provided on the right flank of the dam, as in the plan, to take care of this excess water.

Penstocks

Penstocks, whether made of wood, steel, or reinforced concrete, are meant to carry water from the forebay to the generating station. Very low head plants have the advantage that little or no pipe line is required. Whereas on high head scheme pipes are most often a very serious item of expenditure. Under the ideal conditions, we should use a separate pipe line for each turbo-generator, but we have to consider the economical side also and arrive at a safe compromise between theory and practice. That is why the most practical design often is to use two or more larger pipes of large size in the upper section branching off into individual turbine pipes of smaller sizes and designed for a higher velocity

Fig. 68.



Each pipe line has bifurcated into two parts just near the power-house, each supplying water to one turbine.

in the lower sections. For hillside stations like Kashmir and Jammu, wooden or reinforced pipe can be used, but for medium falls the rivetted type steel pipe line is used and for high head the welded steel pipe line is used. But for very high falls weldless homogeneous pipes are used direct from the ingot. We should try to avoid bends and gradients, but where it is not possible we have to provide for anchorages with a good factor of safety. Thus substantial anchorage is necessary not only at intervals but at bends and where the pipe line has a natural tendency to creep downhill. The diameter of the pipe is designed for a velocity of 3 to 4 ft./sec. on medium heads and 9 to 12 ft./sec. on high heads.

Fig. 69.



Nearer view of Penstock entering power-house.

Anchoring

Anchoring of pipe is done by passing the pipes through masonry constructions at intervals, especially at the head and tail of a bend. These rigid constructions prevent any thrust.

Fig. 70.

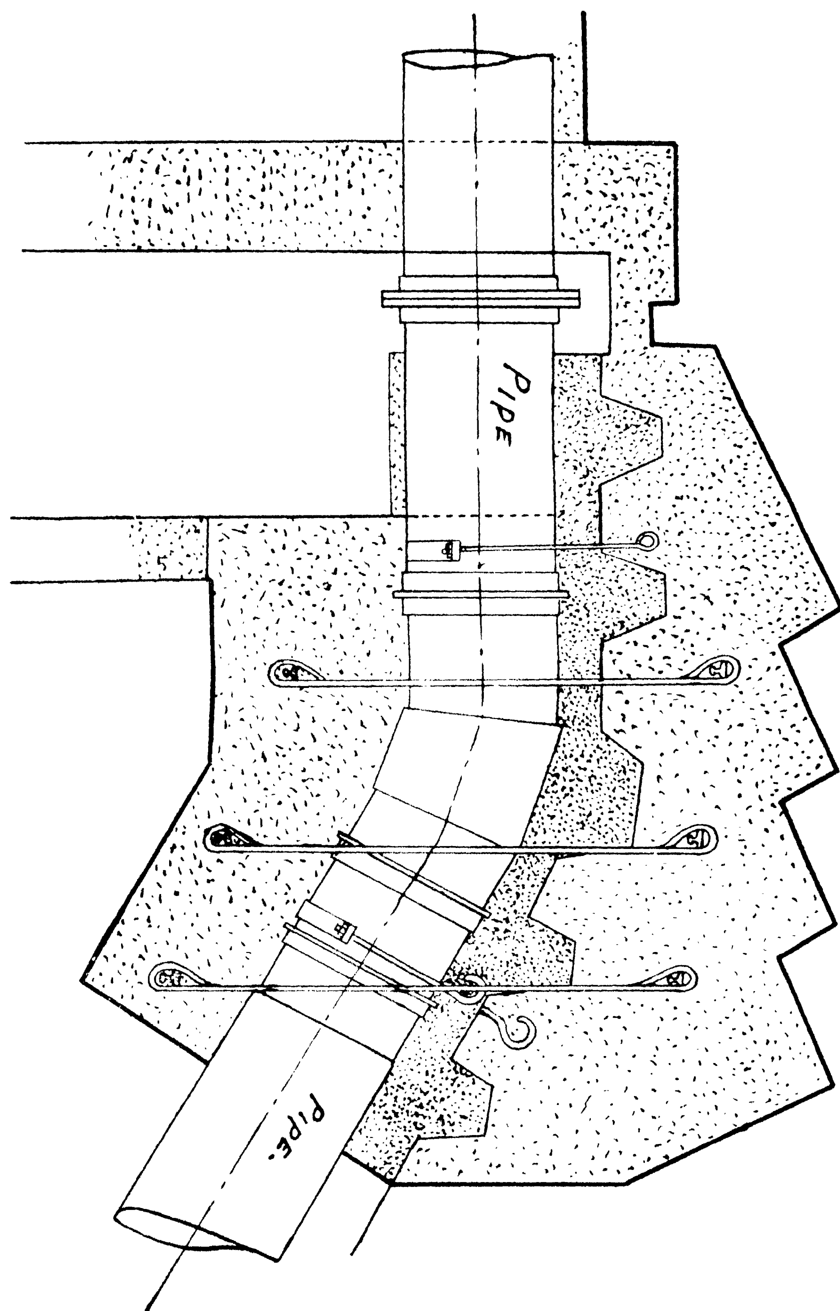
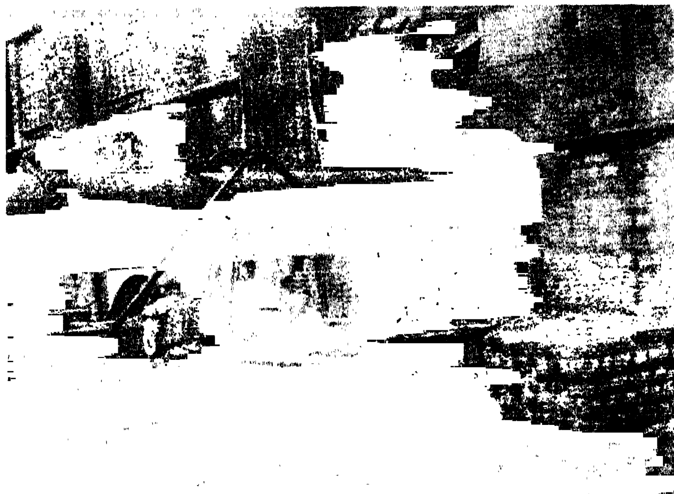


Fig 71.



Nearer view of the Penstock. It is fitted with a sluice valve and similar needle valve on exciter Bus pipe.

Penstock (Cauvery)

The penstocks are thirteen in number, 12 for the turbines, one for each, and the thirteenth for the water-driven exciters. At the forebay the penstocks are equipped with automatic valves that can be closed from the generating station also, when necessary, and will automatically close when the water passing through exceeds a predetermined velocity, operating to protect the generating stations from flooding in case of penstock failure.

Fig. 72.



Exciter Penstock.

The penstocks are made up of rivetted pipe of different diameters and thickness of plates ; the top third is 58″ diameter with $\frac{1}{4}'$ plate, the middle third is 54″ diameter with $\frac{3}{8}'$ plate double rivetted and the bottom third is 51″ diameter with plate $\frac{1''}{16}$, $\frac{1''}{2}$ and $\frac{9''}{16}$ double rivetted.

In designing the penstocks an extra $\frac{1''}{16}$ was added to the thickness of the plates for long life and the average diameter was based on the maximum safe velocity of water and a minimum loss of head. The several diameters were selected to permit three lengths being vested for shipment resulting in a considerable *saving in transportation charges.*

Surge tank or stand pipe

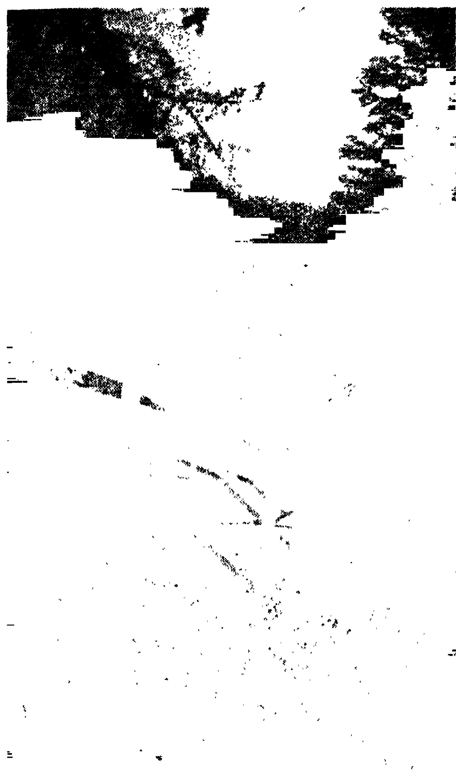
Surge tanks are simply elevated reservoirs connected to the penstock near the forebay or very near the power-house. The latter method of connecting them is more in vogue but, in Sivasamudram they are very near the forebay and fairly high enough above the forebay level, open type and having diameter equal to that of the penstock or the power line. A sudden increase of penstock pressure will cause a rise in the water column in the tank, while a sudden decrease in the pressure will be compensated for by a momentary supply from the tank.

Surge tank.—The simplest method of avoiding the positive water hammer pressure is to provide a by-pass to take the rejected flow. This may be done by (1) installing a relief valve at the turbine, which however effective for gate closure is unable to supply necessary water demanded by the turbine and hence it has no effect on negative water hammer. Or (2) by providing a surge tank at the lower end of the conduit.

Thus a surge tank is invariably provided at the lower end of all very long closed conduits.

Surge tanks may be classified as (1) simple surge tanks—in this water simply flows into the tank when rejected by the turbine. Ordinarily the water will not spill over the top and the tank is made sufficiently large so that the water may not spill over the top under the most drastic condition of load rejection. The action is rather sluggish, it requires the greatest volume and hence it is the most expensive type and is seldom used. The simple tank is ideal as far as ease of governing is concerned.

Fig. 73.



Surge tower on Penstocks, Mahora.

When sudden acceleration of the water has created a temporary vacuum on the pipe line, failure by collapse may follow if air is not immediately introduced to relieve the low pressure region. Hence, the penstock should be protected by air valves of the spring loaded check valve type where abrupt changes in the penstock profile introduces the possibility of a separation of the moving column of water.

It is evident that the *surge tank* performs two important functions. First, it acts as a receptacle for the rejected flow and a reservoir to supply the demanded increment. Secondly, it provides the accelerating or retarding head necessary to correct the flow in the pipe line. Obviously, the more effectively the accelerating or retarding head is applied, the shorter will be the duration of the surge and less water will have to be drawn from or received by the tank resulting in a smaller tank both in diameter and height.

(2) *The Restricted-orifice Surge Tank.*—It is installed between the conduit and the tank. *Objection.*—It creates an appreciable friction loss when the water is flowing to and fro from the tank. The surplus water passes through the restricted orifice and immediately a retarding head equal to the loss due to the restricted orifice is built up in the conduit. The size of the orifice is approximately equal to the ultimate rise of water surface in the tank.

Advantages.—It is more efficient and economical, but on account of the sudden pressure changes in restricted orifice tanks it is unsuitable in places where close governing is required.

(3) *Differential Surge Tank.*—This is a compromise between the simple tank and the restricted-orifice tank. It has an additional

interior riser and for this is distinguised from a simple tank. The internal riser is of a smaller diameter than the connection of the conduit. At the base of the internal riser there is an annular port communicating with the tank. The characteristics of the surge depend upon the area of the port.

In this the initial pressure changes or head of the turbine occurs quickly enough for good efficiency of the tank and is still spread over a period long enough to enable the governors to adjust the turbine gates to compensate the change in head.*

Gates and Valves.—For the Glen Morgan line, as already said elsewhere, only a hand-operated butterfly valve is installed. For the main scheme it is proposed to instal automatic motor-operated butterfly valves with bye-pass at the top of the penstocks and a venturimeter in each line. These valves are designed to close automatically an excess velocity due to a break in the pipe line or similar cause. They can also be closed from the switchboard in the power-house, but it is always safe to operate them from the valve house. At the P. H. a pressure valve of the rotary type will be used.

Valve motors are always rated for the maximum available torque for starting. When D. C. motors are used, they are compound wound with sufficient shunt field to limit the speed at light load. When A. C. motors are used, squirrel cage induction motor with a high resistance to increase the starting torque is used. It may be mentioned here that the torque requirements vary greatly during a cycle. It is maximum shortly after unseating the valve. Afterwards, it is small. While closing, at start only friction has to be overcome, but towards complete closure the pressure causes the torque to increase, but the starting torque to raise the valve gate is the greatest. The motors have to be designed for very rare intermittent action and must be proof against weather or idleness. Limit switches which will open the circuit when the gate has reached its limit of travel are always provided. Such switches are geared to the valve stem and arranged to open the contractors at a predetermined point of the travel of the gates in either direction.

(The capacity of each penstock is 40 cu. secs. dia. at top = 30″, bottom 21″.

Velocity at top = 8′ per sec. ;

Velocity = 17′ per socond (less than 20).

The figures correspond to normal load of 10,000 h. p.).

*For further details and design *vide* Chapter XXV, Hydro-Electric Hand Book by William Creager and Joel D. Justen, John Urley & Sons Inc., New York, Chpaman & Hall, Ltd., London.



Motor-controlled Penstock Main-Gate-operating Mechanism.

Vent Pipes and Air Admission Valves

'Vent pipes' are vertical pipes which are connected to the penstocks and are opened at the top, being, in effect, a form of very small surge tank. These must be sufficiently high to have their upper end 4′ to 10′ above the elevation of the reservoir from which the penstock receives its supply.

The four vent pipes on the penstocks at Mahora are each 12′ high, *i.e.*, their open ends are about 6′ ft. higher than the forebay water-level. There are two relief valves on each pipe line, placed on the inclined portion.

Fig. 74.
View of the valve house, Tata. It is here that the underground tunnel ends.



The pipes in front of the valve house are 58″ diam. and 84″ diam. pipes are at the back of the valve house embodied in the tunnel exit.

If t = thickness of pipe-wall in inches,

d = diameter of pipe in inches, and

p = pressure difference in pipe in lbs./sq. inch,

then the minimum thickness of the pipe strong enough to resist the maximum difference of pressure, is—

$$\text{then} \quad p = 50200000 \times \left(\frac{t}{d}\right)^3 .$$

In the case of the existing penstocks the maximum pressure difference allowed consistent with its strength.

= 98 lbs. per sq. inch.

Even if the weight of the pile of snow on the pipes in winter is taken into account and if the pressure inside the pipes becomes quite low due to the sudden opening of the valves, which is, however, not the usual practice, the pressure difference cannot rise to such figures as 98 lbs. per sq. inch, and so the pipes are quite strong so far as the collapse due to external pressure is considered.

In the case of long penstocks, *i. e.*, where the length exceeds 150 ft., expansion joints should be provided to compensate for

the changes in length due to changes in temperature. There are many types of expansion joints used in practice, the one used in the penstocks here is the *sliding joint*, each pipe-line being provided with one. The joint is placed near the forebay in the horizontal portion of the pipe just below the bend. In this joint the upper section of the pipe fits inside the lower one, which overlaps by about 12″. The change in the length of the pipe-line due to temperature variations makes the upper pipe slide into the lower one. The joint is made water-tight by stuffing jute into it.

The amount of variation in the length of a pipe line in feet is given by $\cdot 0000068 \times$ length in ft. $\times$ range of temp. F°.

In our case :—

$$= \cdot 0000068 \times 550 \times 90$$

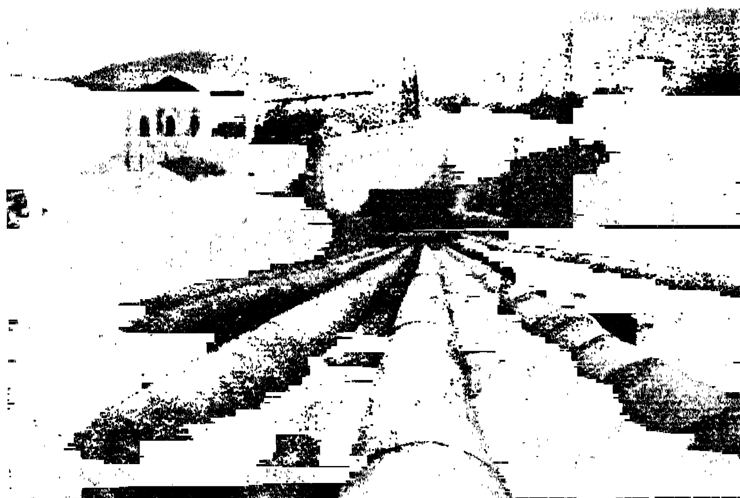
$$= \cdot 033 \text{ ft.} = \cdot 4 \text{ ft. nearly.}$$

Gibson says :—In order to prevent difficulties arising from liberation and accumulation of air at such points and from admission of air at leaky joints, the greatest height above the gradient line should never exceed 20 ft. Owing to the weakness of large pipes under external pressures, such pipes should not be laid above the gradient line.

The power station is located at the dam or the terminal of the diversion conduit. In the first position it may receive the water directly from the upper pool or from a forebay guarded by head-gates, when taking water from the pond the power station stands on the dam alignment ; when fed from a forebay it stands at some angle to the dam, the forebay parallelling it on the land side. In either position the power station is exposed to all the consequences of the fluctuating head and flow, and when on the dam alignment, to those growing out of floatage accumulations.

The power station rests upon a sub-structure whose upstream part, excepting when water enters in pipes, fills the office of a dam. The power station proper is not likely to develop any serious defects unless there be leaks beneath or around it from the forebay.

Fig. 75.



Power station, Khapoli.

The Draft Tube

The suction or draft tube was originally designed with a view of enabling the turbine to be placed at a convenient height above the water-level without loss of head. The maximum practicable elevation depends on the diameter of the draft tube.

The Draft Tube is an air-tight cylindrical extension secured to the lower or discharge end of the runner. The theory of its service is based upon the atmospheric pressure of 14·72 pounds per sq. inch, which equals the weight of a column of water about 34 ft. high; in other words, a column of water of this height, and at rest, is balanced and therefore held in equilibrium by the atmospheric pressure. When the water in this column is in motion, falls, the head represented by the velocity with which it falls at the point of exit from the lower end of the column must also be balanced by the atmospheric pressure, and the theoretic height of the column held in equilibrium, or which can be maintained in a draft tube, is then reduced by this head.

If the exit velocity from a turbine runner is that due to a head of thirty feet or approximately $0{\cdot}2\sqrt{2gh} = 7{\cdot}2$ ft., and the water continues with this velocity to the exit end of the draft tube, the head represented by this velocity is $h = 7{\cdot}2^2 \div 2g = 0{\cdot}8$ ft., and the theoretic height of the column held in equilibrium in the draft tube is $34 - 0{\cdot}8$.

If the velocity in the draft tube were 46·8 ft. per second, then the corresponding head would be $h = 46 \cdot 7^2 \div 64 \cdot 4 = 34$ ft. (about), and the theoretic height of the column balanced by the atmospheric pressure would be zero.

Any other losses of head occurring during the water's passage through the draft tube, as well as that represented by the energy remaining in the finally escaping water, must be deducted from the theoretical 34 ft.

From the foregoing it is apparent that by the use of the draft tube the exit velocity from the runner may be materially reduced during its passage through the draft tube by gradually increasing the flow area in the latter, and therefore the otherwise lost head is conserved and made available to produce useful work.

That the opportunity of gain from this source is greater with a long than a short draft tube.

All the reaction turbines in the station have been provided with draft tubes. This consists in arranging the discharge pipe so that its lower end always discharges below the tail race level. By this means the turbine can be elevated above the tail race level without any loss of head, for since the pressure in the draught tube at the tail race level is atmospheric, the pressure of the draught tube being the same the pressure in the draught tube at the discharge level, *i. e.* (which is not more than 34 feet of water, as atmospheric pressure cannot support more than 34 ft. of water at rest) will be less than atmospheric by the difference in level between turbine and the tail race level. The available head measured from head waters (forebay) to the discharge side of the turbine is the same as if the turbine were placed at the tail race level and discharged into atmosphere without a draft tube. Hence, a draft tube facilitates to erect a turbine above the bottom of head or falls.

Thus the use of the draft tube permits the placing of the *turbine at a convenient height above the tail water*, while without it, the turbine must be placed at or below the lower level.

That the draft tube makes it feasible to place the turbine on a horizontal shaft at sufficient height above the lower pool to allow of directly connecting it to the electric generator or other machinery; and, therefore, it is true that the draft tube brought out the horizontal turbines.

A correct draft-tube design is absolutely essential in order to obtain the maximum efficiency of a turbine as a whole. The fundamental principles underlying their design and construction are that the water shall leave the draft tube with as small a

velocity as possible. The best and most efficient draft-tube design is dependent upon the proper elevation of the turbine above.

It will also be understood from the theory presented that draft tubes *cannot be used, as such, with impulse turbines*, since the water column passing through the impulse turbine is not continuous.

These are generally made of steel or concrete ; they discharge horizontally under water and are easily cushioned ; they are tapered so as to open up gradually and are designed for velocity at exit of about 5 ft. per sec. the mouth being not less than about 2 ft. below low tail water level. All curves should be very gradual, otherwise eddies or the entry of air will cause serious loss of efficiency.

The draft tube can be carried under the generator, if necessary, though it should preferably pass to one side ; and the open tail race begins beyond. Theoretically, the draft head may be equal to the height of the water barometer in the locality, varying, of course, with altitude, but about 34 ft. at sea-level ; actually, it varies with the size of the tube, being greater with small sizes ; *viz.*, from about 29 ft. with a draft tube 1 ft. diameter down to 9 ft. for a 14 ft. tube.

The exit diameter should be 1·5 of the entrance diameter ; the sum of the entrance diameter and the length above tail water should be approximately :—

29 feet for diameters up to two feet,
27 feet for diameters from two to three feet,
25 feet for diameters from three to four feet,
23 feet for diameters from four to six feet,
22 feet for diameters from six to twelve feet.

The pit and tail race :—The pit is arranged in the substructure of the station. The tail race is in the river channel below the power station. They form the exit passage of water. They should be of ample area and depth. The pit and tail race are part of the power station, forming the exit passage of water discharged from turbines. No serious defects are to be expected in either of these when, by proper design, they are of ample area and depth.

When the equipment consists of several units, some of which are operated only for the generation of secondary power from higher than normal flow, it may be found that the tail race under

and from the idle units becomes silted, thereby reducing the depth under draft tubes to such an extent that the outflowing water backs up and reduces the effective head.

The remedy is to operate each unit alternately to avoid long periods of idleness.

The water from the wheels after performing its useful work is led into the tail race, which carries it away either back to the stream from which it came or sometimes to a different one. Where impulse wheels are used, the discharge from the wheel casing to tail race is open and the casing contains air as well as water. Where reaction wheels are used, the casing is entirely filled with water and is continued in the form of a draft tube, which discharges beneath the surface of the tail race thus utilizing by vacuum or suction any further fall (draft head) below the wheel so long as it is within the limits of the water barometer.

The tail race channel has to pass under the floor of the power-house. But it must not pass under the generators.

Tail water velocity usually ranges between about 3·0 and 4·0 feet. Velocity of water in the suction pit ranges between about 1·5 and 2·0 ft.

Tail Race, Jammu

As there are four turbines, there are four draught tubes. Each of these draught tubes discharge in the tail race and below the surface level in the tail race. For this purpose, the tail race has been formed into four parts and each part goes below the power-house building and thence below the turbine pits. Each of the draught tubes discharge in the part below it. All the discharge water joins the main tail race and forms the canal again. The control of the tail race water is by tail race gates. There are four gates in all, one for each turbine. The gates are similar to those used at the head race and the operation of the two is exactly alike.

The usual water-level in the tail race is 7 feet and this level can be slightly controlled by the operation of the gates, but this is seldom done.

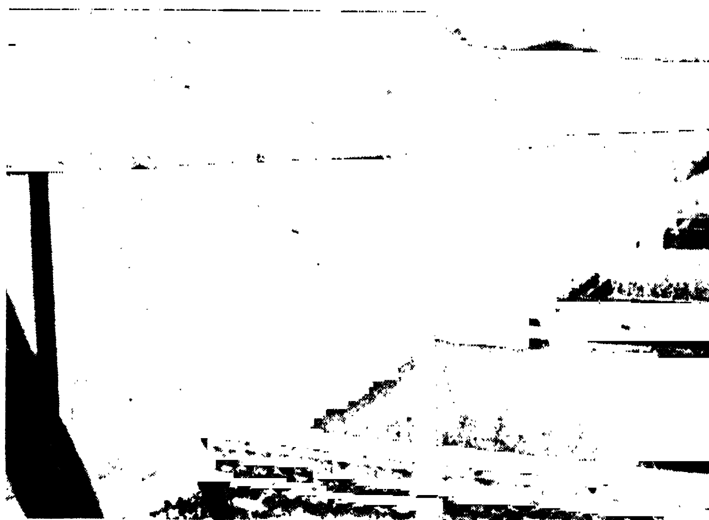
The design of the tail race is slightly defective. A good gradient ought to have been provided in the tail race, at least near the power-house. But the gradient is very little. That is the reason why the water is still in the tail race. It happens sometimes that the draught tubes discharge more water in the tail race than what the tail race can discharge. The water in the tail race, then, rises very much.

Fig. 76.



Back of power-house showing the tail race, Mahora.

Fig. 77.



Construction of tail race for 6000 kW. Shiva Samudram, reaction turbine.

Tail Race, Mahora

Here, the water falls on the Pelton wheel from the jet, and deflected by the deflector hoods. After giving off the kinetic energy to the wheel, the water falls below in the pit, from where it goes to the tail race and thence to the river Jhelum.

The water enters the tail race from five different entrances. The pits of Pelton wheels Nos. 1 and 2 combine and discharge their water from the 1st entrance to the tail race. Similarly, Nos. 3 and 4 Pelton wheels discharge together by the entrance No. 2. The No. 3 entrance to the tail race is meant for the purposes of extension. The fourth is for the combined discharge of the water from exciters, turbo-motor-generator set and the transformer cooling water. The fifth one is also for extension.

The gradient of the tail race is quite stiff and the water flows very swiftly. The size of the tail race is also big as that of the flume, even bigger, and it can carry four times as much water as it carries now. This is necessary to avoid back pressure.

At the place where the tail race meets the river, there is a huge slope in it, which ensures that even if the river level rises, the tail race will work satisfactorily.

CHAPTER IX

Dam

One of the most essential features in the hydro-electric scheme is a dam.

When lakes are not present to offer natural possibilities for effective and relatively cheap regulation of the run-off, it is then necessary to build dams and form artificial reservoirs for impounding the run-off.

The use of dam in Hydro-electric Projects:—Dams are placed across a stream (1) to impound water or to hold up the water for creating or increasing the working head. They may form complete headworks at the intake of a conduit.

(2) To divert a part or whole of the river flow towards the water wheels either by means of head-race flumes or pipes under pressure serving.

(3) As storage or pondage.

(4) To assist in forming intake.

Types of dams and their choice

I. (1) Gravity or solid dams in which the weight of the dam provides the stability, *e. g.*, masonry, earth, rock fill or timber crib for low heads up to 30 ft.

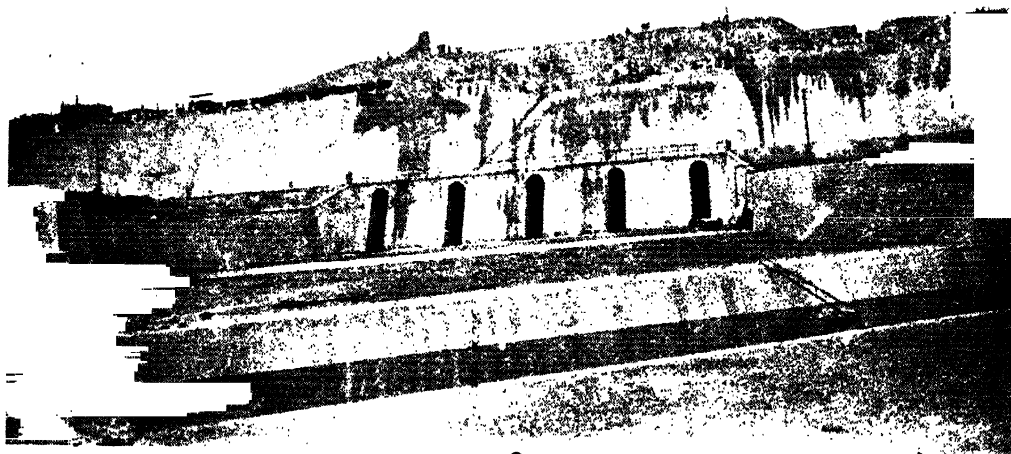
(2) Hollow or buttressed dams.

(3) Arch dams.

II. They may be fixed or movable.

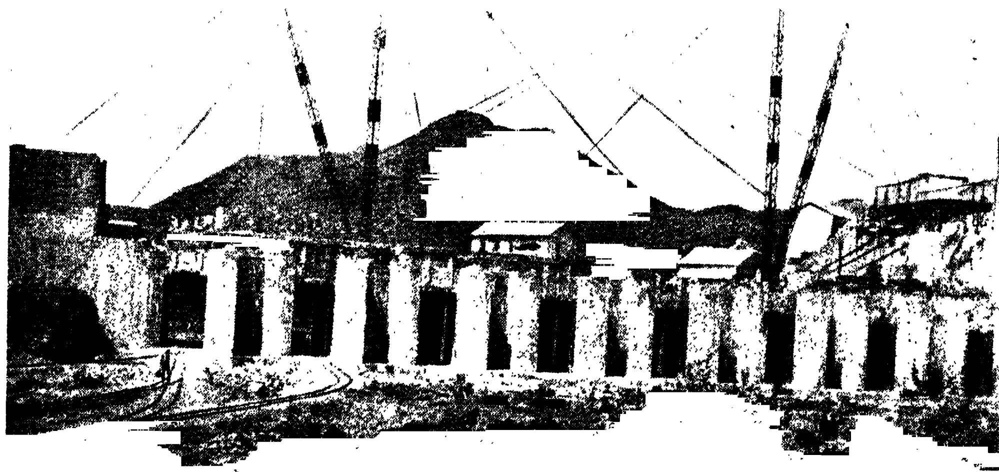
III. They can be built with their crest above the highest waters, and provided with sluice ways, or they can be designed with spillways allowing the water to flow over their crest.

Fig. 1.



Low level sluices, a drop in front Mittur.

Fig. 2.



High level sluices from the lake side, under construction.

Materials of dams :—They can be built with earth, loose stone, timber, masonry, concrete, reinforced concrete and even with steel.

The choice of dams depends on local condition and the profile of the dam site. The nature of the soil in the foundation should be such that there must be a water-tight joint between the materials of the dam and that of the foundation. On rock or shale foundation a masonry or concrete dam may be constructed. The availability of suitable constructional materials and other hydraulic works decide whether masonry or concrete dam is to be made. Want of proper building sand and materials in the vicinity may cripple the development entirely, or greatly increase the cost of it. Thus the accessibility of site as regards transportation facilities and other consideration, etc., are important factors. A poor foundation may prove fatal to the success of a water power project. Test pits or borings may have to be made at the selected site to give a good general indication of the nature of the bed or the like, where it is proposed to build the dam.

Good rock foundations and abutments are always favourable assets. The physical conditions at the site of a dam and the appurtenances are about the most important factors affecting the cost of construction. The best type of dam for a particular location will depend to a great extent upon the nature of its foundation, the profile of the dam site, the materials available for the construction of the dam and other hydraulic works.

Success of a dam will depend upon :—

(*a*) Suitability of the foundation.
(*b*) Capacity for maximum flood.
(*c*) Gates to keep low water head at maximum while providing ample capacity with gates open to keep head and back waters at their designed limits.
(*d*) Strength and stability.
(*e*) How tight the ends and base are against seepage.
(*f*) Provision against erosion downstream from the structure.
(*g*) The provision for fishways, valves or gates.
(*h*) Whether it is long enough for passing floods and short enough to avoid high cost.

Location of Dam:—Investigate the value of storage available and the extent of land and property to be flooded. These affect the quantity of water, which is to be utilized and the height of the dam, and thus they determine the amount of power that may be available. The character of the foundation material

is of the greatest importance. The material must have ample bearing capacity for the load imposed. It must be impervious or of such nature that it can be made so by artificial means. It should be thoroughly explored by test pits and boarings to determine its suitability for foundation purposes.

The proximity of available materials such as rock, sand, etc., is a deciding factor. The facilities for spillways and channels to take care of flood discharge is also an important point.

Make comparative estimates of a number of sites and then determine the most suitable position. Generally, it is found necessary to build a dam or weir across the river at the point of diversion.

* *The economic height of the dam and the contour to which the storage water will be collected behind it will depend upon:—*

(*a*) The requirements for power or irrigation, or both.

(*b*) The catchment area available.

It is frequently possible either to add extra catchment areas, where the run-off is deficient, by means of water-shed tunnels or contour canals, or to divert a portion of the original catchment to another drainage, when it is in excess of requirements and there are difficulties about the escape.

(*c*) The rainfall on, and the actual run-off from, the catchment.

(*d*) The nature of the submerged area, where an extra foot or two of height may involve, on the one hand, the submergence of valuable land, or, on the other hand, an extra 50% or 100% of useful storage capacity.

(*e*) The nature of the seasons. In some localities the annual rainfall is very variable both in amount and intensity, so that the run-off may vary from zero to 80 % or 90 %; here provision must be made by means of large additional capacity for carrying over water from good to bad years.

Where arrangements cannot be made for surplussing, except at the dam itself, under-sluicing is essential with high dams, and the design must ensure that waves caused by high winds down the lake cannot cause the dam to be topped. With low dams, on the other hand, the crest may be the most convenient escape.

* Hydro-Electric Development by J. W. Meares, p. 70 (Pitman's Primers).

The natural stream above and below the dam may also control the height to which the water rises.

The height to which the dam should be built, may be limited by the total cost of the flooded land, or perhaps the topography above the site of dam, may control the height. It will also depend upon the amount of foundation clearance, amount of freeboard, and full supply and high flood levels, *vide* p. 175.

Masonry Structures

General conditions of stability

Masonry structures are generally designed so that there is only compressive stress between the blocks of which the structure is composed. The tensile stress at any part of the structure must not be present. Although mortar has tensile strength to some extent, the usual practice is to assume that it can bear no tensile stress. Moreover, the adhesion between the masonry and the mortar is neglected, and hence the shear or the tangential force acting upon any part of the masonry must not be greater than the natural frictional force between masonry upon the masonry, so that the tangential force may not slide one portion of the masonry upon another. The designer is to see that the weight of the structure must be such that the tangential force may not overturn it on the supposition that mortar can bear no tensile stress. The material of the masonry can bear a compressive stress which has got a limit, and hence the maximum compression stress must be within the safe stress for the material. Hence, the following conditions are to be satisfied in the design of the masonry structure :—

(*a*) There must not be any tensile stress at any cross-section of the masonry.

(*b*) The adhesion between mortar and the masonry is neglected, and hence the shearing force at any part of the masonry must not be greater than its natural frictional force.

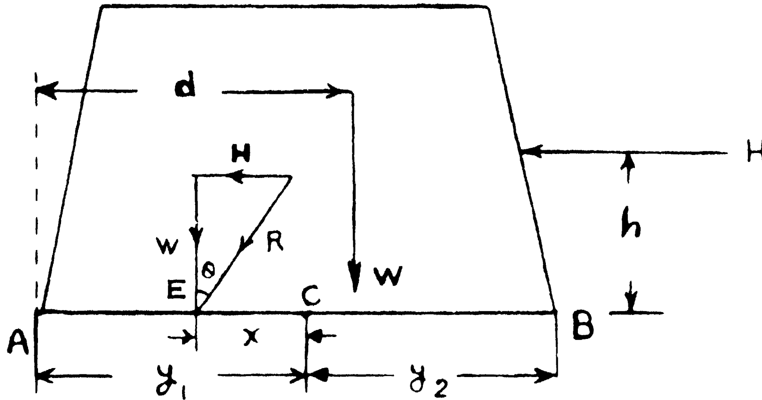
(*c*) The maximum compressive stress must be within the safe stress for the material.

(*d*) The tangential force must not overturn the masonry supposing that mortar can bear no tensile stress.

Condition (a) :—Let AB (Fig. 3) represent the cross-section of a masonry structure having its centroid at C. Let R be the

resultant pressure upon AB, which meets the section AB at E. Resolving R vertically and horizontally we see the section is subjected to a direct thrust W and a shearing force H.

Fig. 3



The direct stress for compression = W/A, where A is the area of the cross-section AB.

The section is also subjected to a bending moment equal to W × CE or W × X.

$$\therefore \quad \frac{WX}{AK^2} = \frac{f'_A}{Y_1}, \qquad \text{or} \quad f'_A = \frac{WXY_1}{AK^2}$$

where f'_A = compressive stress at A due to the bending moment and K is the radius of gyration of the cross-section.

Similarly, $f'_B = \frac{WXY_2}{AK^2}$, where f'_B is the tensile stress at B.

$$\therefore \text{ combined compressive stress at A} = \frac{W}{A} + f'_A$$

$$= \frac{W}{A}\left(1 + \frac{XY_1}{K^2}\right) \qquad \dots (1)$$

and combined tensile stress at $B = f'_B - \frac{W}{A}$

$$= \frac{W}{A}\left(\frac{XY_2}{K^2} - 1\right) \qquad \dots (2)$$

Because, the first condition is that there must not be any tensile stress,

$$\therefore \quad \frac{XY_2}{K^2} - 1 \text{ must be negative.}$$

$$\text{or} \quad \frac{XY_2}{K^2} \text{ must be less than 1.}$$

i.e., the maximum possible value of X is given by—
$= K^2/Y_2$ ... ... (3)

If the cross-section AB be rectangular and AB = b, then $K^2 = b^2/12$, and $Y_1 = Y_2 = b/2$.

whence the maximum possible value of X from (3) is given by—

$$X = \frac{b^2}{12} \times \frac{2}{b} \text{ or } \frac{b}{6}$$

That is, X cannot be more than b/6 on both sides from C. Therefore, E must be within the middle third of the cross-section. This is called the LAW OF MIDDLE THIRD.

Condition (*b*) :—Let μ = coefficient of friction for the material.

$\therefore$ The frictional force $F = \mu W$.

In order that shearing force must not be greater than the frictional force,

H must be less than F

or H ,, μW

or $\frac{H}{W}$,, μ

or $\tan \theta$,, $\tan \phi$

where ϕ is the angle of friction.

Hence θ must not be greater than the angle of friction for masonry.

Condition (*c*) :—It will be satisfied if the maximum compressive stress is less than the safe stress for the material of the masonry.

Condition (*d*) :—Let the horizontal force H be acting at a distance h from AB and the line of action of W meet AB at a distance d from A.

In order that the masonry must not be overturned about A, Wd must be greater than Hh.

Masonry Dam

The dam is the gate-way and key to the power plant and frequently its most important and costly part, and for these reasons it is generally planned and constructed with that care which guarantees for its safety and permanency.

The ideal dam is the masonry dam built on solid rock or hard pan. Such a structure inspires confidence, as it has the manifest appearance of strength and as a dam is generally intended to be a permanent construction, a masonry structure is invariably the best to resist the action of the elements for centuries. They are generally built with a practically vertical upstream face and a downstream slope of 1 to 1. These are generally used with storage dams with sluice ways.

Classification of Masonry Dam

The masonry dams are divided into the following three general types :—

(1) Solid gravity dam.
(2) Hollow gravity dam.
(3) Arch dams.

Merits compared :—Hollow dam contains about 35 to 40% of the concrete required for a solid dam, but it is considerably more expensive per cubic yard. Hollow dam is cheapest for remote localities where the transportation cost for cement is great.

The solid gravity dam is economic for localities near a rail-road where the ingredients for concrete aggregate are near at hand. Arch dams are the least expensive, but the site should be adaptable to this.

The dam should be made as nearly homogeneous and monolithic as possible, interlocking the stones in all directions. Cutstones are not suitable for the masonry dam.

The solid gravity and large arch dams may be composed of ashlar masonry, mortar rubble masonry or concrete. Large stones are usually imbedded in the concrete for these types. The mix is usually in the proportion of 1 : 3 : 6 throughout.

The Forces Acting on Dams

The following forces act on a masonry dam :—

(*a*) Water pressure including uplift and impact.
(*b*) Earth or silt pressure.
(*c*) Ice pressure.
(*d*) Weight of the dam.
(*e*) Reaction of the foundation.

If the fall is to be created by means of a dam, the height h to which it can be carried is generally determined by the level of the country above which it may be flooded and the nature of the soil on which the foundation is laid.

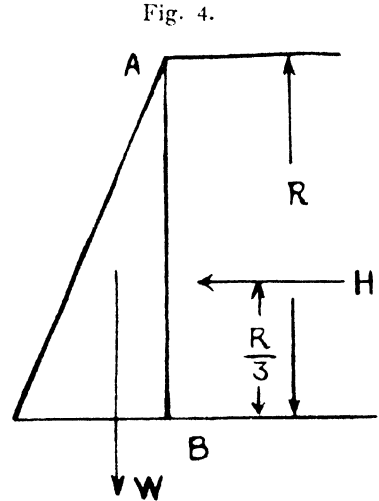
The Fig. 4 represents a dam whose vertical rectangular plane AB of area A sq. feet is under water pressure. Let b be the width of the plane. The plane AB is subjected to a horizontal pressure H, when—

$$H = A \frac{h}{2} W = 62{\cdot}5 \frac{bh^2}{2}$$

where W = weight of 1 cu. ft. of water, which is 62·5 lbs.

This pressure H is acting at the centre of pressure of the plane area AB and which is at a vertical distance h/3 from its base.

Fig. 4.



Water pressure.

The impact of water approaching against the upstream face of the dam—

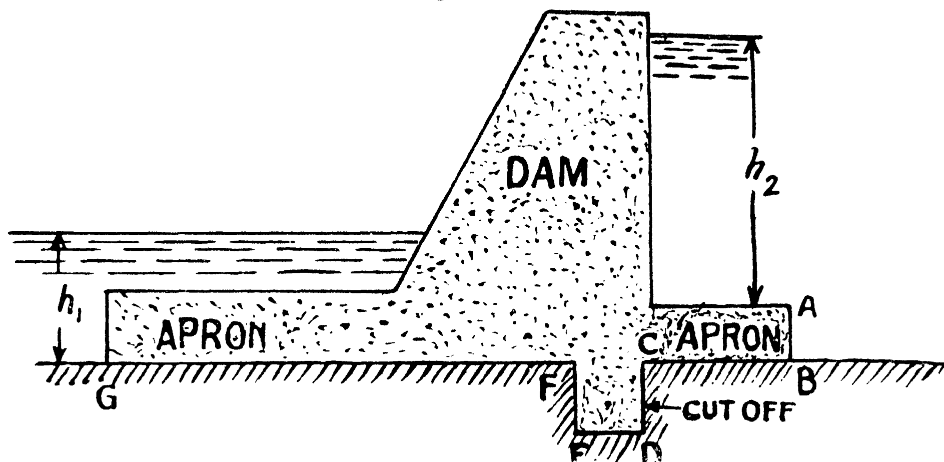
$$\begin{aligned} &= \text{loss of momentum per second} \\ &= \frac{62{\cdot}5\, b\, h\, v}{g} \\ &= \frac{62{\cdot}5\, b\, h\, v}{32{\cdot}2} \\ &= 1{\cdot}93\ b\ h\ v. \end{aligned}$$

Where v is the average velocity of approach in feet per second in the middle of the channel. This pressure can be considered to act at a distance of $\frac{h}{2}$ from the base. The impact is only to be considered for low dams having large discharge.

Uplift:—It is the upward water pressure acting at the base of the dam. There is head-water on one side of the dam and tail-water on the other side. Due to the difference of the head of water on the two sides of the dam there is a flow of water under its base. Hence the base of the dam is subjected to an upward pressure.

It has been proved by experiments that the flow of water under dams has pressure characteristics exactly similar to the flow of water in pipes. The difference $h_2 - h_1$ is the head lost in friction. But the friction loss varies directly as the length of the path of percolation between the masonry and its foundation. Hence $h_2 - h_1$ is lost due to the path of percolation which is to be made to such a length with apron and cut-off as shown in Fig. 5,

Fig. 5.



so that the difference of head $h_2 - h_1$ may be completely lost in friction.

Length of path of percolation = AB + BC + CD + DE + EF + FG = X, say.

∴ head lost in friction per unit length of the path—

$$= \frac{(h_2 - h_1)}{X}.$$

∴ uplift pressure at $C = h_2 - \frac{(h_2 - h_1)(AB + BC)}{X}.$

Similarly uplift pressure at

$$F = h_2 - \frac{(h_2 - h_1)}{X}(AB + BC + CD + DE + EF).$$

In the case of rock foundations it is assumed that only a percentage c of the base is subjected to uplift pressure. If the foundation has no cut-off, the resultant uplift on the base may be expressed by the equation :—

$$W = 62{\cdot}5\, c\, b\, l \times \frac{h_1 + h_2}{2}$$

where, l = horizontal length of the dam.
b = its width.

This resultant pressure can be located at a distance X from the heel when—

$$X = \frac{l\,(h_2 + 2h_1)}{3\,(h_2 + h_1)}.$$

The value of c is 0·66 for rock-foundations, but for ordinary structures a lower value is adapted. For earth foundations c is unity.

The uplift pressure being more at the upstream end of the base a special effort is often made to obtain greater resistance at the stream end or bed of the base by the following methods :—

(1) The first method is to increase the resistance at the heel. For this reason trenches are often excavated at the heel and then refilled with masonry, or holes may be drilled in the rock and grouted to provide an efficient cut-off. This is the reason for providing a cut-off at the heel of the dam.

(2) The second method is to decrease the resistance below the heel. For this reason drainage systems are to be constructed between the dam and the foundation, downstream from the heel, to allow free exit of the water that passes the heel.

Uplift should be considered to exist on all the horizontal points in the masonry above the base.

In hollow dams it is assumed that there is no uplift due to head-water as the cavities in the structure effectively relieve the buttresses of all such joints.

Fine silt will generally be deposited by the stream against the upstream face, being washed down by floods. Similarly, considerable quantities of sand and gravel are also deposited against the upstream face of the dam. If there be no special sluicing provisions to prevent their accumulation, it is reasonable to assume that uplift from head-water will not exist. But the stability of the dam should be tested with and without the silt pressure.

Ice Pressure :—Ice being lighter than water will float upon it. Hence, an overturning effect will be experienced upon the dam due to the expansion of ice when the temperature changes. It is impossible to determine the thrust of ice to its exact value. In Europe values varying from 25,000 to 48,000 lbs. per linear foot of the dam are generally taken. But in India the formation of ice is seldom experienced and hence the ice pressure can be neglected.

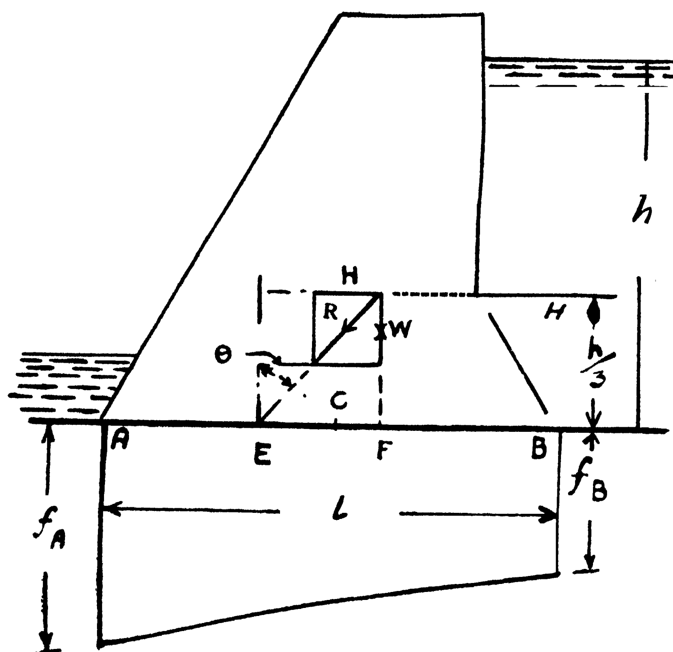
At the time of flood no ice pressure usually exists. It is, therefore, seldom necessary to design for ice pressure when water surface is above the level of the spill-way crest.

The Weight of the Dam:—The weight of the masonry depends upon the ingredients of which it is composed. It varies from 145 to 150 lbs. per cubic foot. In important structures the value should be determined from the weight of the masonry to be used. The weight of the dam can be considered to act at its centre of gravity.

Causes of Failure and their Remedies

(1) There must be no tensile stress at any point of the section:—

Fig. 6.



In order that there must not be any tensile stress at any point of the rectangular section AB of the base, the pressure R, which is the resultant of the weight W of the dam, and the water pressure H must be within the middle third EF of the section AB when the dam is full of water. And if it meet AB in E when $AE = \frac{l}{3}$, the compressive stress at A is, therefore, given by:—

$$f_A = \frac{W}{A}\left(1 + \frac{CE.AC}{K^2}\right) \quad \dots \quad \dots \quad (1).$$

$$= \frac{W}{A}\left(1 + \frac{l \times l \times 12}{6 \times 2 \times l^2}\right)$$

$$= \frac{2W}{A}$$

$$= \frac{2W}{bl} \quad \ldots \quad (2).$$

where, A = area of the rectangular section AB
= b × l.

The compressive stress at B—

$$F_B = O.$$

If the resultants meet at any other point between EF, the compressive stresses at A and B can be determined from the equation (1) and (2) above ... ...

Another condition to be satisfied for this is that the weight W must not meet the base AB outside EF when the reservoir is empty.

In the case of spill-way dams it is impossible to provide suffi cient masonry at the top to force the resultant within the middle third of upper horizontal joints. In such cases tension is allowed if it is not great; although in some cases steel reinforcement has been provided.

Regarding the maximum inclined stresses cosiderable disagreement exists among engineers. They do not use the exact determination. From the following equations the maximum value can be otained :—

At the heel B of the dam,
maximum compressive stress $= f_B \sec.^2 \theta$.

At the toe A of the dam,
maximum compressive stress $= f_A \sec.^2 \theta$.

(2) *The maximum compressive stresses must not exceed safe limits* :—The maximum compressive stresses in the dam and in foundation should not exceed allowed limits. Their values must

be within the safe bearing power of the soils. The following table gives the best of the safe bearing power for different soils :—

Kind of Material.	*Safe bearing power in tons per sq. ft.*	
	Minimum.	Maximum.
Rock—the hardest in thick layers in native bed	200	...
Rock equal to best ashlar masonry ...	25	30
Rock equal to best brick masonry ...	15	20
Clay in thick beds always dry	5	10
Clay in thick beds moderately dry ..	2	4
Gravel and coarse sand well-cemented ...	8	10
Sand compact and well-cemented ...	4	6

At the base the compressive stress is maximum at the toe A where its value is f_A sec. $^2 \theta$.

(3) *It must resist the sliding effect of water* :—In order to resist the sliding effect of water, it is customary to provide that the frictional resistance alone shall be sufficient to resist the resultant of all the horizontal forces with ample margin of safety, as it is customary to assume that the adhesion between the masonry and the mortar is neglected. The frictional resistance is μW, where μ is the coefficient of friction, and this frictional resistance must be greater than the horizontal pressure H. In practice the general formula used is :—

$$\mu W = SH \text{ or } \tan\theta = \frac{\mu}{S}$$

where, S is the factor of safety desired.

For rock foundations S can be taken as unity. If the dam be anchored with deep cut-off walls or piles, S also may be taken as unity. For earth foundations without cut-off S must be more than 2·5.

Material.	μ
Granite on gravel and sand (wet) ...	0·41
Pine on gravel and sand (wet) ...	0·41
Granite on sand (dry)	0·65
Point-dressed granite on like granite ...	0·70
Point-dressed granite on common brickwork	0·63
Point-dressed granite on smooth concrete ...	0·62

Material.	μ
Fine cut granite on like granite ...	0·58
Dressed hard limestone on like limestone ...	0·38
Dressed hard limestone on brickwork ...	0·60
Common bricks on common bricks ...	0·64
Common bricks on dressed hard limestone ...	0·60

It is usual to assume a value for masonry on masonry and for masonry on rock lying between 0·6 to 0·75.

(4) *It must not fail by overturning* :—The dam will overturn if the moment of H about A is greater than that of W about the same point and when the tensile stress will be produced at B. This condition is generally satisfied if the first condition is satisfied, still we are to verify this. The equation generally adopted is :—

$$W\ AF = S.\ H\ h/3$$

where S is the factor of safety against overturning. The value of S must not be less than 2.

(5) *It must be guarded against possible temperatures cracks, and also against the effect of ice expansion* :—In large masonry dams the interior gradually acquires the average temperature of the year and changes but little throughout the year In the winter the outer portion of the masonry and specially those exposed to the air contract both horizontally and vertically, and as the temperature remains nearly constant in the interior, it does not contract, the outer portions will crack. Thus tensile stresses will be produced in them so that these portions of the dam become ineffective for conveying compressive stresses to the foundation. This feature is an important one to include in the computations for a dam.

The largest mass of concrete is below, at the location of the spill-ways. But experience shows that concrete is so poor a conductor of heat that the centre of a large mass does not fluctuate in temperature in harmony with the atmosphere, but remains of one stable temperature all the year round. Hence, it is necessary to provide for temperature expansion and contraction only at the surface and for a certain distance into the mass. In this work the expansion joint of tar paper on edge is made at one side of each spill-way in each span and from the face the width of the tar paper into the mass of concrete. The tar paper is placed on edge between the spill-way section and the pier, following the curves and vertical upstream face of the shape of the spill-way.

(6) *The masonry must not be crushed* :—The experiences of various year has abundantly determined that particular care ought to be taken with the foundations on which a dam rests. Most dams fail due to settling, foundations washing out, slips, percolation water seeping along the masonry and earth division, muskrats, excessive leakage through the foundations. For the sufficiency of the rock foundation the engineer is to think first. After excavations have been carried down to a stratum that has the surface appearance of being entirely satisfactory, this surface is then washed with steel brooms and water under pressure in order that all the foreign matter may be located and removed from the surface of the rock. Concrete dams should be built in alternate blocks and not as a continuous wall. Otherwise shrinkage cracks may occur because the concrete always cracks when setting in air.

Expansion joints are provided against contraction.

Another most important problem that confronts the engineer in planning the construction of a dam is the method of taking care of the river during the construction. The diversion is generally effected by means of timber cribs or rock fill coffer dams which are to be designed to withstand overflow in case an extraordinary flood occurs during the construction period.

(7) *The dam must be protected against leakage under its foundations* :—The leakage most generally appears under a dam or around the abutments. It is very difficult to find the remedy for a leaky dam which is founded on rocks. The location of the leak must first be determined. This is invariably found through a rock stratum with its heading in the river bed possibly at a considerable distance upstream. To locate the origin of the leak it will become necessary to lower the water level and the pond to examine the shallow flowing water. This will reveal the point by the bubbling of the water into the rock crevice, which marks the upstream opening of the defective stratum. When the leak is found, one of the several treatments may be employed to fill the fissure with some impervious material, such as perfectly cement grouting which must be forced into it with some pressure. Water flow must, of course, be excluded for a sufficient time to permit the filling to set. Small leakage under the dam which is founded on alluvial material may be arrested by depositing ashes along the upstream face, which will be washed into the openings which may thus be closed. When this is not effective, sheet piling may be restored to,

and if this fails to correct the defect, the water of the pond must be drawn down for the construction of the concrete cut-off wall which is made lower than the leaking point. The super-structure will develop leaks only when cracks open in it because of settlement or temperature influences. Leaks around abutments are probably more frequent than in other locations, because abutment and embankment cores are not carried to sufficient depths or are found on porous material.

Design of a Solid Gravity Dam

The design of a non-overflow masonry dam is always started at the top. The super-elevation of the top of the dam above high-water surface is generally made sufficient to get beyond the reach of waves and to provide sufficient weight to resist ice thrust. It is generally not more than 5% of its maximum height.

According to the *Thiery's Formula* the width b at the top of the dam,

$$b = \frac{h}{12}$$, where h is the height of the dam.

But the value of b is usually within 10 to 14% of the maximum height unless a greater width is required for a roadway or other purposes.

The top of the dam having been fixed, the design proceeds taking proper care for the cases of failures.

The *Thiery's Formula* also gives an approximate determination of the width at the base as follows :—

$$B = \frac{h}{2}\left(\frac{2}{3} + \frac{1}{2}\right)$$

where B = width at the base.
h = height of the dam.
But it must be tested with the middle third rule.

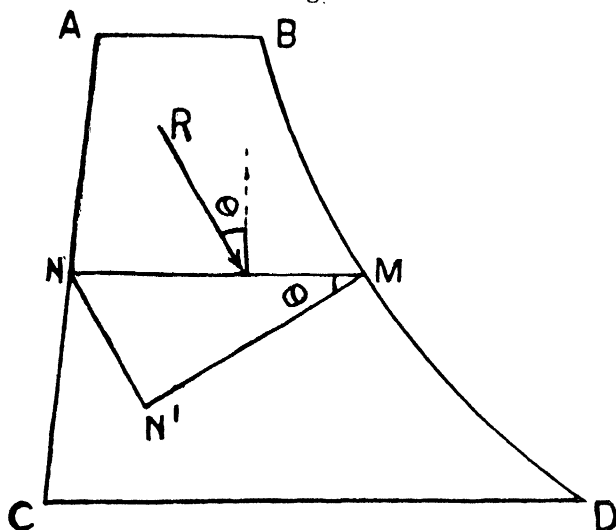
In order to obtain the maximum stress at any point in the masonry the *Bouvier's Method* is used in practice. Let ABCD be the profile of a dam (Fig. 7).

Let R be the resultant of the weight,
w of the dam and the water pressure H,
θ be the angle which the resultant R makes with the normal.

Therefore, the force R is distributed along MN and the stress is maximum at M.

According to *Bouvier* the force R is to be distributed along MN′, where MN′ is equal to MN cos θ in order to get the maximum stress at M, *i.e.*, the maximum stress at M

Fig. 7.



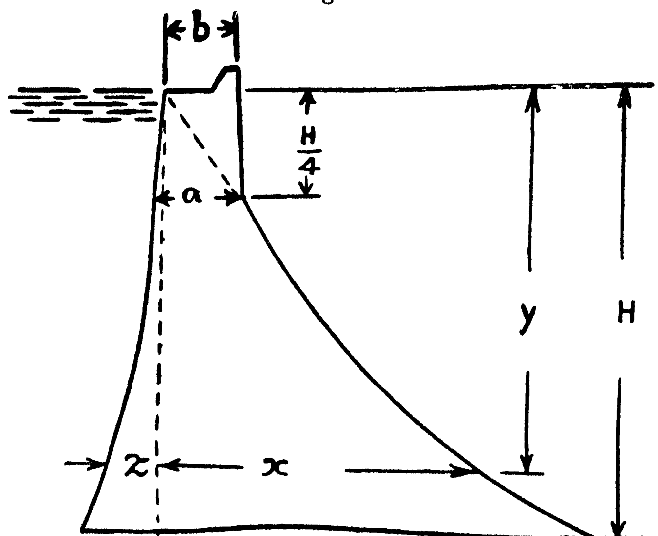
$$= \frac{R}{b.\ MN'}$$

$$= \frac{R}{b.\ MN \cos \theta}$$

where b = width of the dam at the section MN.

Lavy's Method is another principle adopted for getting the compressive stress at any point along the upstream face. He says the compressive stress at any point in the masonry must not be less than the water pressure at the same point and that if any line is drawn across the profile, there be no tensile, no crushing nor sliding stress at any point along the said line.

Fig. 8.



Molesworth Formula is the another practical rule for the design of a dam.

The formula is

$$y = \sqrt{\frac{0{\cdot}05\ x^3}{P + {\cdot}03\ x}}$$

$$z = \left(\frac{{\cdot}09\ x}{P}\right)^4$$

b = 0·4 a where a = width of the dam at ¼ H from top.

where, x = depth in feet of a given point from the top.
y = horizontal distance in feet from such a point to the flank or the downstream face of the dam.
z = horizontal distance in feet from such a point to the upstream face of the dam.
P = safe stress in tons per sq. ft. on the masonry.

Forms of Sections :—Actual dams are rarely trapezoidal, but have the flank hollowed and the base widened by a rake of the flank, and just so much on the face as will keep the thrust wall within the middle third when the reservoir is empty.

The water face of the dam is usually so little curved in its vertical section that the water pressures on either part or whole may be taken as if the face were plane. The curve used in general is 1 in 12.

Having adjusted the width at the top which is between 10 to 14 % the maximum height, and the *super-elevation* which is within 5 % of the maximum height, the design proceeds from the top as shown in Figs. 12and 14 so that the line of the resultant pressure (reservoir full) is within the downstream extremity of the middle third and the same (reservoir empty) within the extremity of the middle third.

The width of the zone II is so adjusted that the resultant pressure upon it may be within the middle third of the joints, both faces remaining vertical.

In the zone III the downstream face must begin to batter as indicated so that the middle third rule may be satisfied, upstream face may be kept vertical if the middle third law be satisfied for the reservoir, when empty.

In the zone IV both the downstream and the upstream faces are battered and similar is the case for the other zone so that the middle third rule is satisfied for both the cases, *i.e.*, when the reservoir is full or empty.

At every stage we are to see that tan θ may be within ·7 to ·75. The causes of failure No. 3, *i.e.*, tan $\theta = \mu/S$, are applied only for the zone I, which is rectangular, and for the other ones the value of tan θ may be within 0·7 to 0·75.

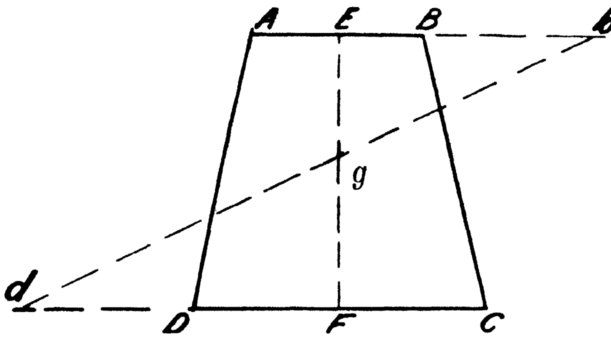
Applications of Graphic Statics

(1) To find the c. g. of a trapezium ABCD :—

Draw Bb = CD

Dd = AB

Fig. 9.



E and F are the middle points of AB and CD, respectively. Join db and EF which will intersect at g, which is the centre of gravity of the trapezium.

(2) To find the c. g. of two trapeziums :—

Let W_1 and g_1 be the weight and c. g. of ABCD.

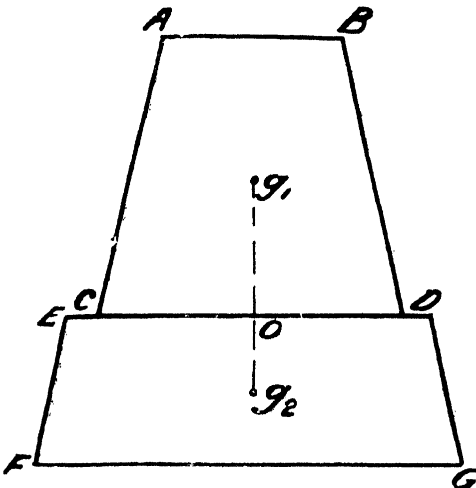
W_2 and g_2 be the weight and c. g. of EDFG.

Draw $g_1a = W_2$

$g_2b = W_1 \times$ (Fig. 11).

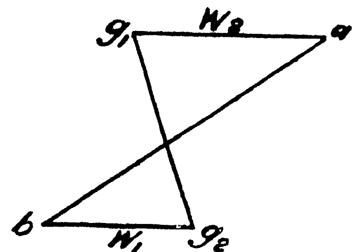
Join ab which meets $g_1 g_2$ at O, which is the centre of gravity of ABDEFG.

Fig. 10.



(3) To find the resultant of a system of parallel forces is the most useful application of the graphic statics in masonry dam in order to find the combined c. g. of the zones.

Fig. 11.



Example :—Design the profile of a non-overflow masonry dam to be erected against a water head of 95 ft. Take 1 cu.ft. of the masonry weight 150 lb. The coefficient of friction for the masonry upon the masonry may be taken 0·75.

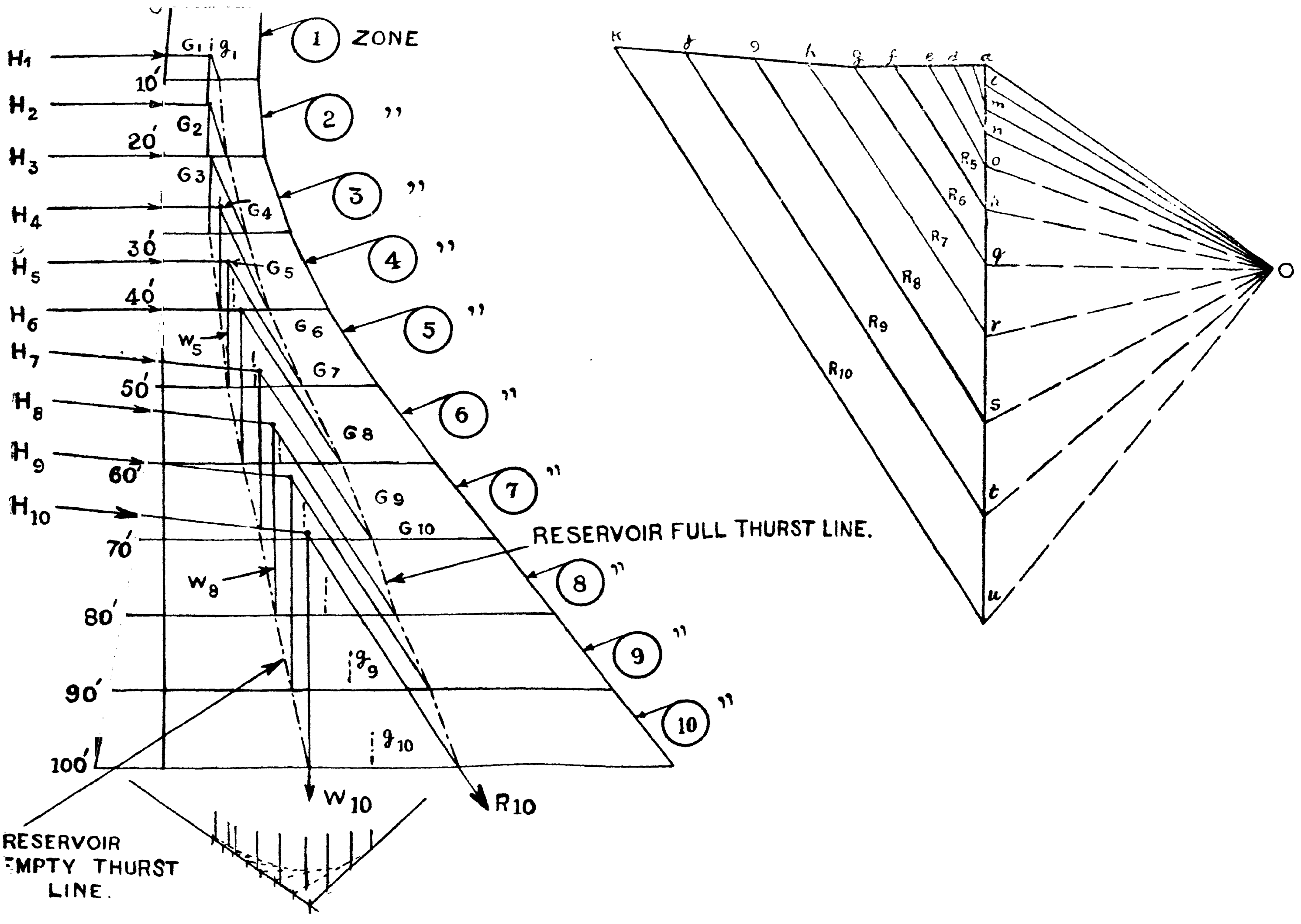

ZONE
RESERVOIR FULL THURST LINE.
RESERVOIR EMPTY THURST LINE.
H_1
H_2
H_3
H_4
H_5
H_6
H_7
H_8
H_9
H_{10}
10'
20'
30'
40'
50'
60'
70'
80'
90'
100'
W_5
W_8
W_{10}
R_{10}
R_5
R_6
R_7
R_8
R_9
O

Fig 1:

(a) Super-elevation of the dam above water level = 5 ft. (say).

(b) Total height of the dam = 95 + 5 or 100 ft.

Hence, the super-elevation is 5 % of its maximum height.

(c) The width at the top = 12 ft.

(d) Take 10 ft. to be height of each zone and therefore the number of zone is 100/10 or 10.

(e) The number of joints = 10.

(f) Considered length of the dam for calculation = 1 ft.

The design has been carried on as if the whole dam with super-elevation is under water pressure. The width at the base of each zone has been selected so that the resultant pressure lines, both when the reservoir is full and empty, are within the middle third. The calculation has been made as follows :—

For zone No. I :—Its base width = 12′.

Earth pressure $= \frac{12 \times 10 \times 150}{2240}$ or 8 tons.

Water pressure $= \frac{62 \cdot 5 \times 10^2}{2 \times 2240}$ or 1·4 tons.

For zone No. II :—Base width = 13′.

Earth pressure $= \frac{12+13}{2} \times \frac{10 \times 150}{2240}$ or 8·4 tons.

Water pressure $= \frac{20+10}{2} \times \frac{10 \times 62 \cdot 5}{2240}$ or 1·7 tons.

For zone No. III :—Base width = 16′

Earth pressure $= \frac{16+13}{2} \times \frac{10 \times 150}{2240}$ or 9·8 tons.

Water pressure $= \frac{30+20}{2} \times \frac{62 \cdot 5 \times 10}{2240}$ or 7 tons.

Proceeding similarly the result has been tabulated as follows :—

Zone No.	Width at base in feet.	Earth pressure in tons.	Representation in the diag.	Water pressure in ton.	Representation in the diag.	Load scale.	Linear scale.
1	12′	8	al	1·4	ab	1″=48 tons	1″=16′.
2	13′	8·4	lm	1.7	bc	,,	,,
3	16′	9·8	mn	7	cd	,,	,,
4	22′	12·7	no	10	de	,,	,,
5	28′	16·7	op	12·6	ef	,,	,,
6	37′	22	pq	15·4	fg	,,	,,
7	45′	27·5	qr	18·2	gh	,,	,,
8	55′	34	rs	21	hi	,,	,,
9	64′	36·5	st	24·2	ij	,,	,,
10	74′	42·7	tu	27	jk	,,	,,

In the diagrams g_1, g_2, g_3, etc., are the centres of gravity of the 1st, 2nd, 3rd, etc., zones, respectively. W_1, W_2, W_3, etc., are the weights of the masonry above the joints at depth of 10, 20, 30 ft., etc., respectively, from the top.

$\therefore$ W_3 = weight of (1st zone + 2nd zone + 3rd zone).
= 8 + 8·4 + 9·8 = 26·2 tons.

Similarly, the others—

$\therefore$ W_{10} = weight of the whole masonry dam.
= 218·3 tons.

H_1, H_2, H_3, etc., are the water pressures upon the portions of masonry above the joints at a depth of 10, 20, 30 ft., etc., respectively, from the top acting at a distance of $\frac{1}{3}$ of the depth of the respective portions from the respective joints.

$\therefore$ H_3 = water pressure on the (1st + 2nd + 3rd zones).
= 1·4 + 1·7 + 7 = 10·1 tons.

Similarly—

H_{10} = total water pressure upon the dam.
= 138·5 tons and acting at a distance of 100/3 or 33·3 ft. from the base of the dam.

H_1W_1, H_2W_2, etc., meet at G_1, G_2, etc., respectively, H_{10}, and R_{10} at G_{10}.

R_1, R_2, R_3, etc., are the resultants of W_1H_1, W_2H_2, W_3H_3, respectively.

For the whole portion of the dam :—

R_{10} = H_{10} + W_{10} = 268 tons.

The factor of safety against overturning :—

$$\frac{W_{10} \times 48}{H_{10} \times 33{\cdot}3} = \frac{218{\cdot}3 \times 48}{138{\cdot}5 \times 33} = 2{\cdot}3$$

$$\tan\theta = \frac{H_{10}}{W_{10}} = \frac{138{\cdot}5}{218{\cdot}3} = 0{\cdot}63.$$

Maximum stress according to Bouvier's principle :—

$$= \frac{268}{74 \cos\theta} = \frac{268}{74 \times 0{\cdot}84}$$

$$= 4{\cdot}3 \text{ tons per sq. ft.}$$

The values of W, H, θ and the maximum intensity of stress at different points have been tabulated as follows from the stability diagram of the dam :—

Distance from the top.	Width of the joint.	Weight of masonry of the portion above the joint in tons.	Water pressure on the portion above the joint in tons.	Resultant pressure in tons.	Tan θ, i.e., H/W	Intensity of stress according to Bouvier' principle.
0	12′	—	—	—	—	—
10′	12′	W_1 = 8	H_1 = 1·4	R_1 = 8·2	0·175	·7 ton per sq. ft.
20′	13′	W_2 = 16·4	H_2 = 3·1	R_2 = 18	0·19	1·5 „ „ „
30′	16′	W_3 = 26·2	H_3 = 10·1	R_3 = 30	0·38	2 „ „ „
40′	22′	W_4 = 38·9	H_4 = 20·1	R_4 = 45	0·52	2·4 „ „ „
50′	28′	W_5 = 55·6	H_5 = 32·7	R_5 = 66	0·588	2·8 „ „ „
60′	37′	W_6 = 77·6	H_6 = 48·1	R_6 = 92	0·62	3·1 „ „ „
70′	45′	W_7 = 105·1	H_7 = 66·3	R_7 = 136	0·63	3·8 „ „ „
80′	55′	W_8 = 139·1	H_8 = 87·3	R_8 = 168	0·62	4 „ „ „
90′	64′	W_9 = 175·6	H_9 = 111·5	R_9 = 214	0·63	4·2 „ „ „
100′	74′	W_{10} = 218·3	H_{10} = 138·5	R_{10} = 268	0·63	4·3 „ „ „

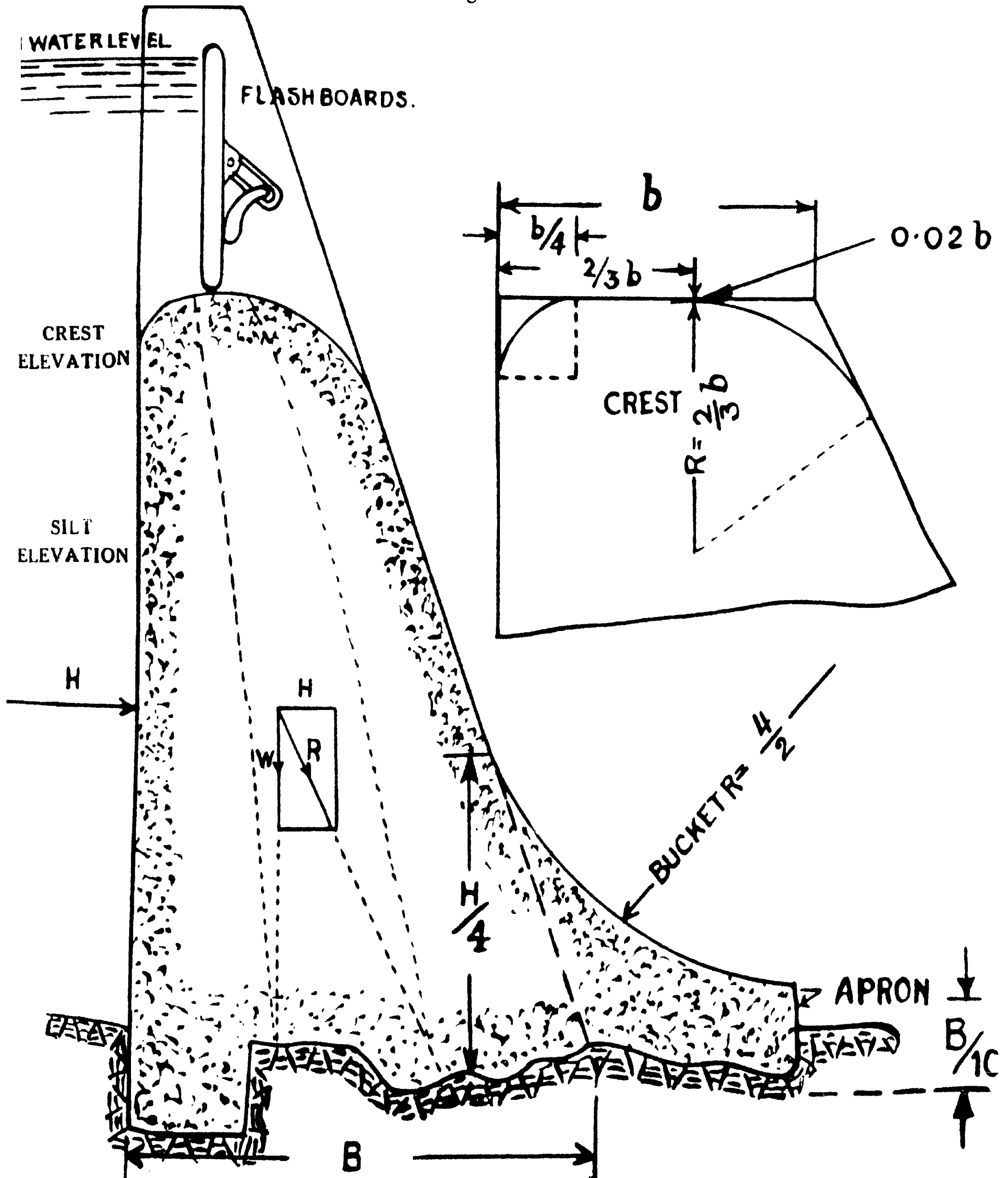
Spill-way Dam :—This sort of dam is frequently used to-day in connection with low and medium head developments on rivers and where large flood discharges must be passed. The following important points are to be considered in the case of a spill-way dam :—

(*a*) The size of the spill-way must be sufficient to take care of the larger known floods. It is generally designed for 10 to 15% greater discharge capacity than any previous record would show to have taken place.

(*b*) The downstream face should be so curved that the water will follow the surface and prevent the formation of a vacuum. This curve will cause the

water to be discharged in a horizontal direction so that the bed of the stream is protected against undercutting and erosion at the lower end of the toe when severe floods are passed and permitting a quiet discharge without subjecting the masonry structure to dangers from vibrations.

Fig. 13.



Generally, for the spill-ways it contains a downstream apron and a bucket to support the spilling water.

(*c*) The top-width should be equal to one-fifth of height of the dam.

(*d*) The safe strength of the materials cannot be taken more than ten tons per square foot.

(*e*) With the maximum flow the factor of safety against overturning must not be less than two.

(*f*) The design must be governed by the other conditions applied to solid masonry dam, *i.e.*, the law of middle third rule, law of sliding friction, etc.

(*g*) The maximum overflow does not exceed two-tenths of the height of the spill-way.

(*h*) The length of the spill-way is given by :—

$$l = \frac{\theta}{h \times v}$$

where, θ = volume of water in cubic feet to be passed per second.

h = height of the spill-ways in feet.

v = velocity of the flow of water in feet per second.

According to Muller formula for the economical minimum shape for a spill-way dam,

$$B = \frac{3\sqrt{hx}\,[x + H] + hx}{4x}$$

where, B = width of the base (toe not considered).

H = height of spill-way dam in feet.

h = height of overflow in feet.

x, y = width of the parabolic face of the dam.

$y = 1{\cdot}5\sqrt{hx}$

Another formula giving the depth of the joint in which the resultant of water and masonry forces leaves the middle third is the following :—

$$y^2 - 3{\cdot}8hy - 12{\cdot}8\,h^2 = 0$$

which gives the condition $y = \frac{x^2}{4h}$.

In practice the Muller formula is not used. The shape of the crest, of the downstream face and of the toe of a spill-way dam are computed as follows :—

In order to prevent the lodgment on or against it during the low overflow of floatage, the crest of the upstream edge should be slightly rounded without giving it an up-slope of any length. The radius of the round is generally one-third the top width.

From the downstream end of this round of the upstream edge, a downward slope of one inch per two feet is given.

The downstream crest is a curve whose radius is two-third of the width at the top touching the downstream slope line and the top width at a distance two-third of it from the upstream face as shown in the figure.

Then for the downstream toe, increase the base by 1/10 of its length horizontally and then vertically by the same amount. Draw a curve with a radius of half the height of the spill-way to touch the downstream face slope line and the highest point of the vertical line drawn. This gives the shape of the bucket of falling water.

This process of construction gives the best form of the apron and the bucket to support the spilling water.

The following elements enter into the computations of the stability of a spill-way dam in the above equations.

(1) Water pressure for static conditions neglecting the height of the overflowing water.

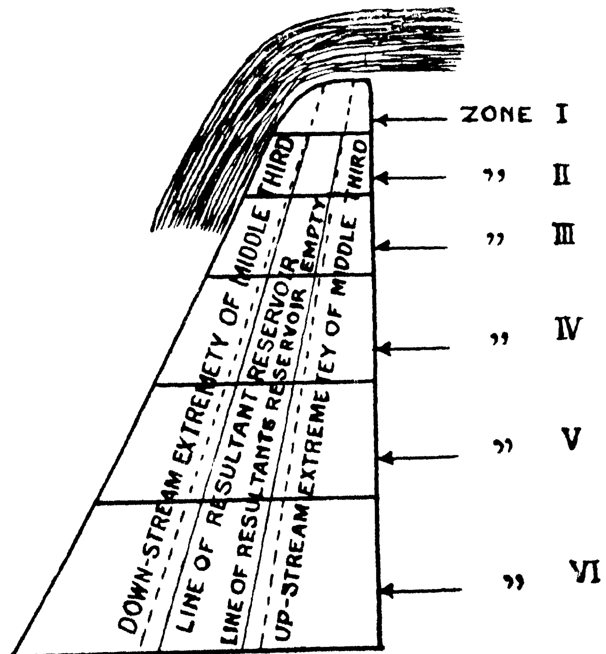
(2) The weight of the masonry acting by gravity alone.

(3) The possibility of vacuum effects, under the overflowing sheet is eliminated, the parabolic face, the dam cutting into the falling sheet.

In designing a solid spill-way dam similar process is adopted as is done for a non-overflow masonry dam. In zone No. 1 the middle third rule and the law of sliding must, of necessity, be violated as it is impossible to provide sufficient masonry to withstand sliding by friction and to provide the resultant within the middle third near top of a spill-way dam.

In the zone No. 2 the middle third rule may be satisfied for the reservoir, both full and empty, but the law of sliding is not satisfied.

Fig. 14.



If the tensile stresses in the above zones seem to be more, the steel reinforcement may be required to resist the tensile stresses and particular care should be taken with horizontal building joints to provide numerous large projecting stones or special key ways to increase the shearing resistance.

For the remaining zones all the conditions for the stability hold good. Finally the dam is to be tested to satisfy the remedies of all the cases of failures as explained in the case of a solid masonry dam.

With the common spill-way, the rate of discharge per unit length depends upon the depth of water on the crest, increasing as the depth increases ; an overfall spill-way may, therefore, not have much discharge capacity until the water surface rises to a considerable height. Hence an overfall spill-way is not immediately responsive to variations in the stream flow, and does not closely control the water level.

In order to increase the discharge capacity the head must be increased thus giving water greater velocity. This is the basic principle of the siphon spill-way. It consists of passages through the dam as shown in the Fig. No. 28, page 221, the lower end of the siphon on the downstream side generally being submerged. If the air be exhausted from the siphon, the maximum head available being the total head between the water surfaces above and below the dam will be useful and a rapid flow will take place. Automatically the air will be forced out of the siphon as soon as the upper water level rises above the air vent sealing it against the admission of air.

The water will then spill over the crown of the siphon forming a diaphragm of water which seals the upper part of the siphon against the entrance of air from bellow. As the flow increases, all the air will be rapidly forced out and the crown and siphon are completely filled with water.

The headwater surface falls if the full discharge of siphons at the dam is greater than the total flow into the pond. When the upper parts of the air inlets are exposed, the air drawn in by the suction reduces the efficiency of the siphons until the discharge is automatically diminished to that required for stationary water surface in the pond. If, however, two much air is drawn in, the siphon action will be broken and the headwater surface will then rise again and the operation of priming will be repeated. If properly proportioned, the siphons will prime within a few seconds after the water has risen to the required elevation.

The inlet is generally made two or three times the area of the throat and is well rounded and is usually protected by rack bars quite wide apart.

The upper leg is made of sufficient length to bring the inlet well below water surface in order to prevent the entrance of ice and drift. The lower leg should be as long as is practicable up to the siphon limit in order to take the advantage of the head available. The outlet may be submerged or may be opened to the air except for special cases where submerge is necessary.

The discharge of a siphon may be computed as follows :—

$$Q = CA\sqrt{2gh}$$

where, h = gross-head on the siphon in feet from the headwater surface to the centre of the outlet.

A = the area of the throat in square feet.

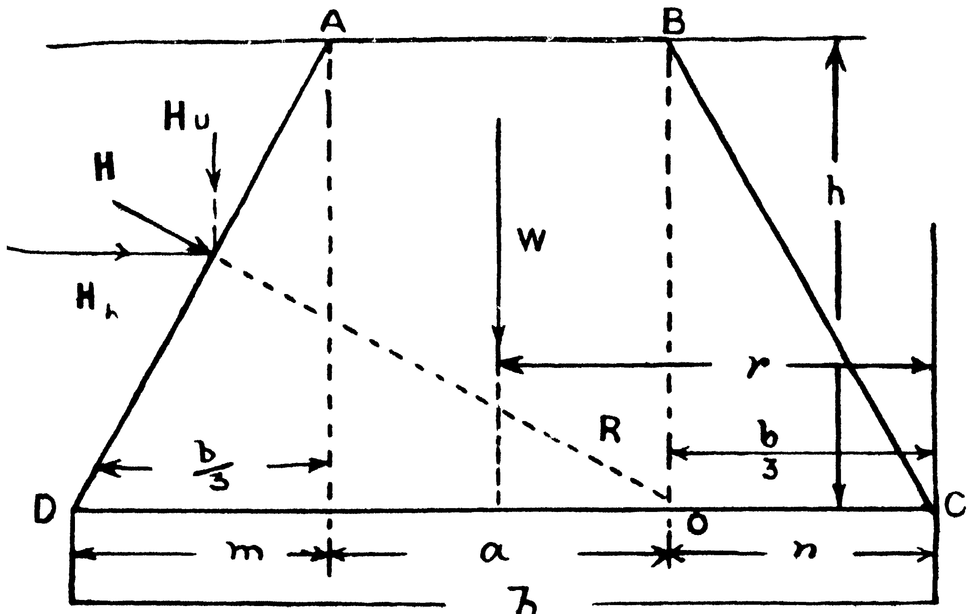
g = acceleration due to gravity.

Q = discharge in cubic feet per second.

c = coefficient depending upon the characteristics of the siphon.

Low Dam :—Suppose ABCD be a low dam. In order to find the condition of its stability only the middle third rule is to be satisfied. The dam being low, other conditions will automatically be satisfied. Still it is to be tested for the law of sliding.

Fig. 15.



H_h = horizontal component of the water pressure H.
H_v = vertical component of the water pressure.
W = force acting at the c. g. of the dam.
S = specific gravity of the masonry.
W_1 = weight of one cu. ft. of masonry.

$$H_v = \frac{62{\cdot}5 \cdot m \times h}{2}.$$

$$H_h = \frac{62{\cdot}5 \times h^2}{2}.$$

The force acting at c. g., i.e., $W = W_1 h \left(\frac{a+b}{2}\right)$.

$$r = \frac{\frac{2}{3} n^2 + a^2 + 2an + \frac{1}{3} m^2 + m(a+n)}{a+b}.$$

R = W + H.

Hence, the dam will be stable if R passes through O, *i.e.*, if the moment of the forces about O be zero, or if—

$$H_h \times \frac{h}{3} - H_v \left(\frac{2b-m}{3}\right) - W\left(r - \frac{b}{3}\right) = 0.$$

The width of the base is therefore :—

$$b = \sqrt{h^2 - (a+n)^2(S-1) + n^2S + \frac{n^2S^2}{4}} - \frac{nS}{2}.$$

where, $S = \frac{W_1}{62{\cdot}5}$.

If BC be vertical, n = 0 and

$$b = \sqrt{h^2 - a^2 (S-1)}.$$

If AD be vertical, m = 0 and

$$b = \sqrt{5/4\, a^2 + \frac{h^2}{S}} - \frac{a}{2}.$$

In practice it is economical to make AD vertical rather than BC.

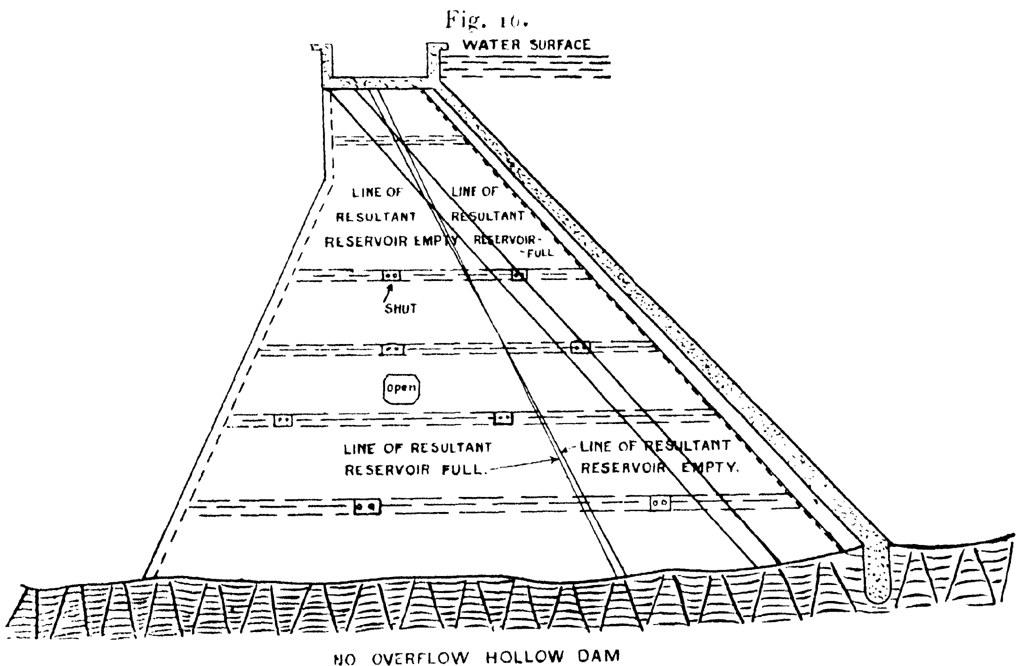
Molesworth gives the following rules for a low dam :—

Width at the bottom = ·7 × height.
Width at the middle = ·5 × height.
Width at the top = ·3 × height.

Hollow Dams:—It requires less concrete in comparison to a solid dam, but it is considerably more expensive per cubic yard. It is cheapest for remote localities where the transportation cost for cement is great. Hence this type of the dam has been devised to utilize the material more economically than is sometimes possible in a solid structure. It is a hollow structure consisting of a reinforced concrete water-tight deck on the upstream side supported at intervals by buttresses or piers perpendicular to the axis of the dam. For spill-ways it contains a downstream apron and bucket to support the sheet of spilling water.

This type of dam is also called reinforced concrete dam.

Fig. 16.



NO OVERFLOW HOLLOW DAM

These dams can be used either on a rock foundation or on certain ground; in the latter case they are generally built over a reinforced concrete mattress distributing the pressure over the foundation.

Their upstream face is generally inclined at an angle of 45° and the buttresses are practically vertical on the downstream side.

In the case of a spill-way the downstream side is formed in the same manner as the upstream side by a reinforced concrete apron of the shape required by the quantity of water which is expected to flow over the crest.

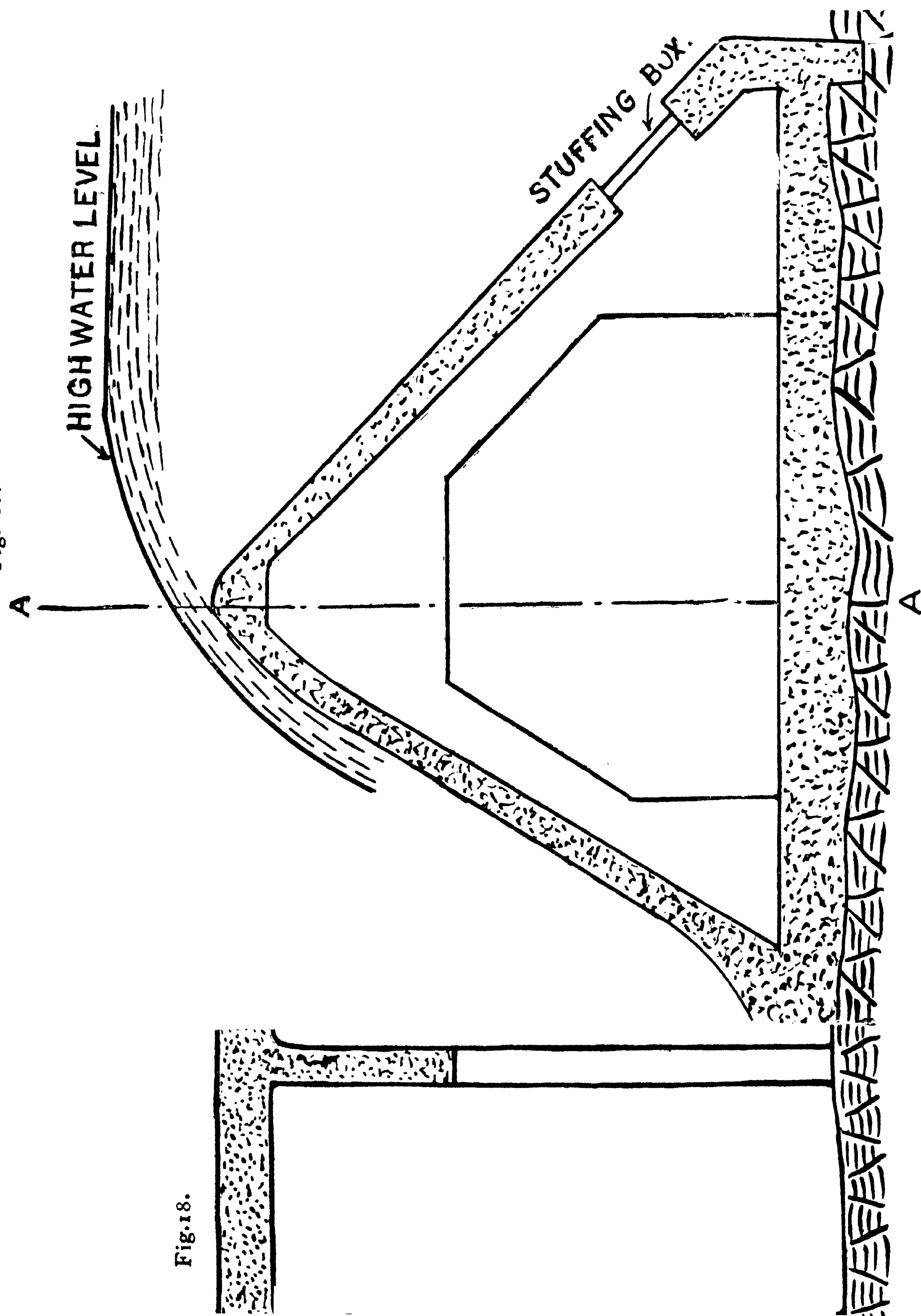


Fig. 17.

Fig. 18.

Hollow Dam, Flat-Deck Type

The following points are to be considered in the design of a hollow or buttressed dam :—

(*a*) The general theory of design for a solid dam, as explained before, will also apply to hollow dams.

(*b*) As the downward pressure of the water is relied on to a great extent to give the structure stability, the upstream face should have a slope of 45 to 60 degrees with the horizontal, thus allowing the utilization of the vertical pressure of a large column of water to assist stability.

Authorities differ as to the inclination of the upstream face.

According to Dr. Gibson's opinion, the slope should be between 40 and 50 degrees to the horizontal.

According to Rushmore, the slope must not be more than 45 degrees.

According to Creager, the slope should be between 45 to 60 degrees with the horizontal.

(*c*) The thickness of the deck should vary uniformly from base to top and be proportioned in accordance with the water pressure it is to resist. It must be reinforced with steel in order to increase its strength.

(*d*) The downstream ends of the buttresses for a non-overflow hollow dam are made vertical or slightly battered. This batter usually starts from any point between the middle to three-quarters-height of the dam up to the base. The batter is generally used to such an extent that the middle third rule may be satisfied.

(*e*) For spill-way hollow dams, the downstream apron and the bucket to support the sheet of spilling water should be designed in the similar process adopted for solid spill-way dams.

(*f*) The structure, specially the deck, must be water-tight.

(*g*) There should be drains and passage-ways for interior inspection.

(*h*) The super-elevation above the water surface and the width at the top for a hollow non-overflow dam is fixed accordingly as outlined for solid dams.

For spill-way dam its shape and crest of the upper part of the downstream face must be sufficient to spill the maximum flood water.

(*i*) The foundations of this type of dam must be of good rock.

(*j*) The buttresses are spaced from 15 to 25 ft. centre to centre having been supported by struts provided at intervals. The buttresses are to be designed in the form of thin walls, quite heavily loaded and well reinforced for stiffness with steel bars extending into the deck.

(*k*) The struts are to be reinforced for compression, tension and also for bending due to their own weight. They are usually spaced centre to centre about twelve times the thickness of the buttresses.

(*l*) Open holes in the buttresses are convenient for the passage of men and material during construction. If the dam be high, an inspection gallery is to be provided.

Classification of hollow dams :—The hollow dams are of two types in general :—

(*i*) Flat-Deck Type.

(*ii*) Multiple-Arch Type.

Arch decks are designed in normal slices and for each slice the crown is at a higher elevation than the haunch, due to which the loading will be less per square foot at the crown than at the haunch. But the top parts of the multiple-arch dams are usually provided with vertical decks. The arches must be reinforced for stiffness and well-tied to the buttresses. A central angle of 120° is most economical for the arches.

The buttresses of the flat-deck type are spaced from 15 to 25 ft. centre to centre. But the buttresses of the arch-deck type are spaced from 30 to 40 ft. centre to centre and this is considered most economical.

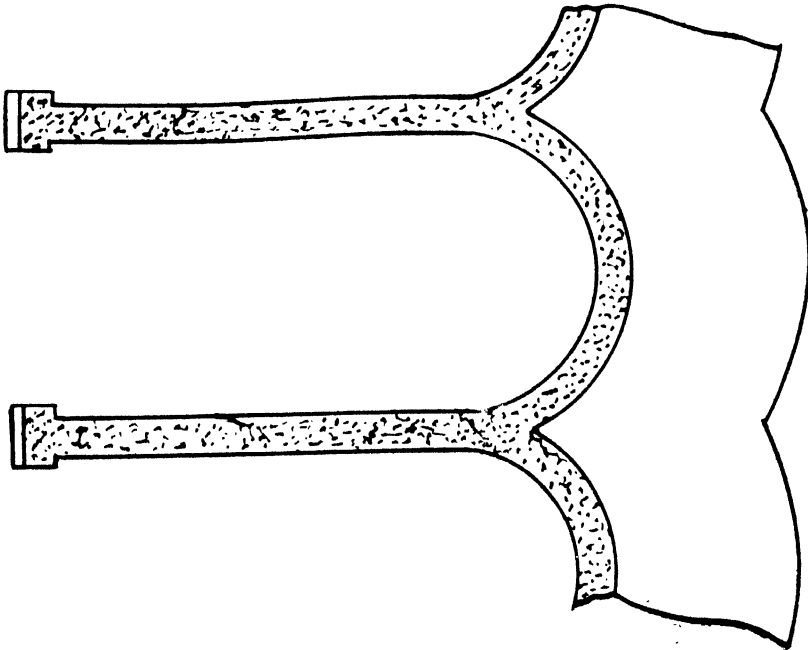
Arch Dam:—These types of dams are only suitable for comparatively short spans in comparison to height. The dam of this type is circular in plan with the convex face up the stream. The thrust due to water pressure is transmitted directly to the sides of valley and hence the sides of the valley are to be composed of good rock to resist the arch thrust at the haunches. The dams of this class are designed as a combined arch and gravity type as it is not a good practice to rely entirely on the action of the arch. In practice it is designed, in many cases, purely as a gravity structure, and the strength of the curved form is simply assumed to increase its safety.

The figure 17 belongs to the flat-deck type class where the deck is flat.

In the multiple-arch type, the deck consists of a series of arches as shown in Figs. 19 and 20.

Fig. 19.

Fig. 20.



The necessary thickness at any depth is given by—

$$R_1 = R_0 \sqrt{\frac{S-H}{S+H}}$$

Where, R_1 = outer radius of the arch in feet.
R_0 = inner radius of the arch in feet.
S = working stress in tons per square foot.
H = water pressure in tons per square foot.

This formula is applied where the arch is considered a portion of a thick cylinder.

But in practice most dams have been designed in accordance with the following equation which applies to these, submerged empty cylinders :—

$$T = \frac{RH}{S}$$

Where, T = thickness of the arch in feet.
R = mean radius of the arch.

The weight of the dam and its attachment to the foundations increase its stability and reduce the stress appreciably, and if the above formulæ be used, the working stresses to be used are much higher than in a gravity dam. The values of the working stresses to be used for different materials are as follows :—

Ashlar masonry (granite)	... 45 tons per sq. foot.
Ashlar masonry (hard sand-stone)	... 28 tons per sq. foot.
Ashlar masonry (soft sand-stone)	... 18 tons per sq. foot.
Concrete (depending on aggregate)	... 15-25 tons per sq. foot.

Mr. Jorgensen has shown that the greatest theoretical economy is obtained when the central angle is 133° at all elevations.

Arch Dam :—Early arch dams were constructed mostly with a constant radius at all levels necessitating a central angle which gradually decreased towards the base of the dam. In cases where the configuration of the rock surface is such that a constant angle would result in less thickness at the bottom than at the higher, it is not possible to use a constant angle. Hence it should be designed to the nearest constant angle as practical considerations will allow.

The uplift pressure at the base is resisted by the weight of masonry which is not included in the forces resisting overturning.

The downstream face of the constant-angle arch dam is usually vertical, but in case of large arched dams the base is thickened by providing a downstream batter in order to safeguard against excessive vertical compressive stresses due to vertical beam section. The rock at the abutments should be well-stepped to withstand the arch thrust safely without sliding at the arch haunch.

Multiple-Arch Dam :—The present practice is to use several arches instead of a single arch; the buttresses being used to convey the downstream thrust of the water to the foundation, the side thrust being transmitted to the valley sides—*vide* Figs. 18 and 19.

Earth Dam

This type of dam generally has a trapezoidal cross-section and consists of an earth-fill faced with some border material. Earth dams can, on account of their large base, be built on less resisting foundations than those constructed of other materials if the lead of river be impermeable. This type of dam is generally used for medium heads of water. It has never been used for heights above 200 feet.

In order to design this form of dam, the engineer is to see that the material, specially gravel, sand and clay, which goes into the structure, must be found near the site of the dam. The materials used in this type of dam must be impervious and the imperviousness of the material used in the construction determines the stability of the dam. The sections of these dams vary according to the local circumstances and the materials used. In general, the upstream slope is either 3 to 1 or 2 to 1, and the downstream slope about 2 to 1. The centre part of the earth dams is always constituted by a core of impermeable material, generally a mixture of sand and clay. Sometimes the puddle wall is replaced by a concrete wall driven into the bed of the river foundation to an impermeable strata. The successful construction of an earth dam requires just as careful attention to details as the construction of other engineering structures. At one time, it was looked upon as merely a fill of dirt. Due to some avoidable failures, thorough investigations and studies are made of the materials and site of this type of dam.

Criteria of the Design of Earth Dams

Criterion 1.—An earth dam should be designed so that the spill-way capacity is so great that there is no danger of overlapping or being washed away.

In many cases the earth dam fails due to the insufficient capacity of the spill-way. A masonry dam, with insufficient spill-way, can stand overtopping to a considerable depth without any serious damage. But the overtopping of an earth dam is its total failure. Another point is to consider that if flood-water be allowed to flow over the crest, its material will be disintegrated and washed away, and hence earthfill dams must be protected against overflow by masonry spill-ways of suitable capacity. If it is necessary to discharge flood-waters over the dam, the spill-way made of masonry must be provided at a lower level than the crest, so that under no circumstances water may flow over the crest. It is often preferable to build an independent spill-way discharging through a tunnel excavated around the flank of the dam with stepped discharging channel for safety.

The structure must be at least 10 feet higher than the high water level.

Criterion 2.—The line of saturation must be well within the downstream toe.

The line of saturation is the uppermost line of flow of the water through the dam and sub-soil. This line of saturation is generally coincident with the hydraulic gradient, which is a line joining the highest points to which water would rise in a series of pipes placed in the cross-section, *i.e.*, the hydraulic gradient is a line joining the heads of water at different points on the cross-section.

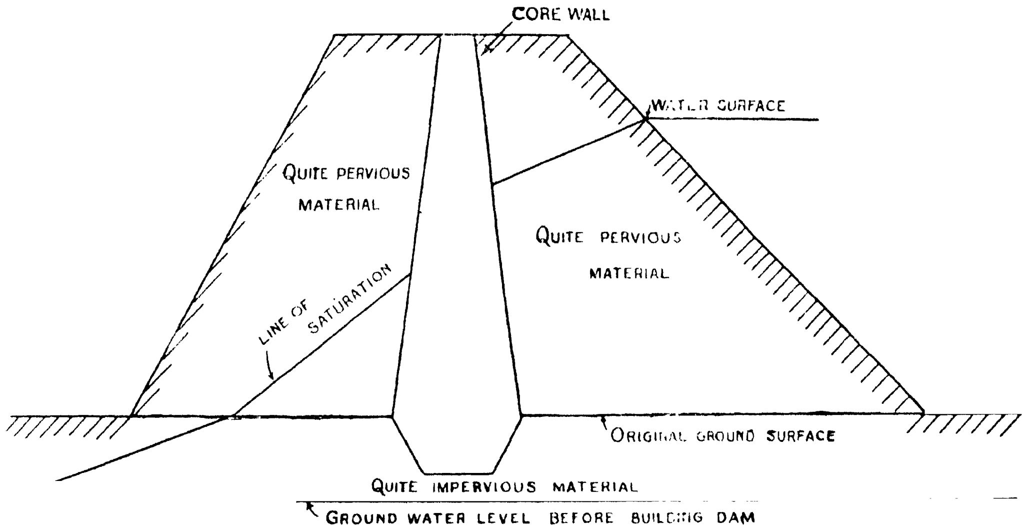
The following factors effect the line of saturation:—

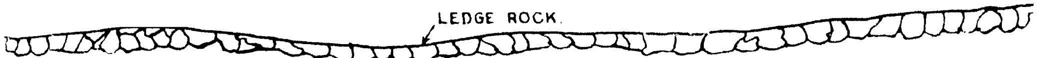
(*a*) Porosity of the soil of the dam and its foundation.
(*b*) Homogeneity of the materials composing the dam.
(*c*) The existence of a core-wall, puddle-wall, etc., with the dam.
(*d*) Depth of soil below the base of the dam.
(*e*) Distribution of the particles of the materials of various effective sizes throughout the crosss-ection.
(*f*) Collection of seepage with drains in the downstream part of the dam.

The line of saturation drops uniformly in the impervious material, but it is much steeper in a pervious material. The upstream part of the dam is to be built with impervious clay, and the downstream part with the pervious material. By this arrangement, the upstream part keeps as much of water as possible out of the dam, and the downstream part drains away as rapidly as possible such water as passes through the upstream part, and thus the stability of the dam is increased.

Another interesting phenomenon is that the line of saturation drops more, if a core-wall is made in the middle of an earth dam. The core-wall is generally made of puddle or of masonry, and it is assumed to be impervious. The effects are explained diagramatically as follows :—

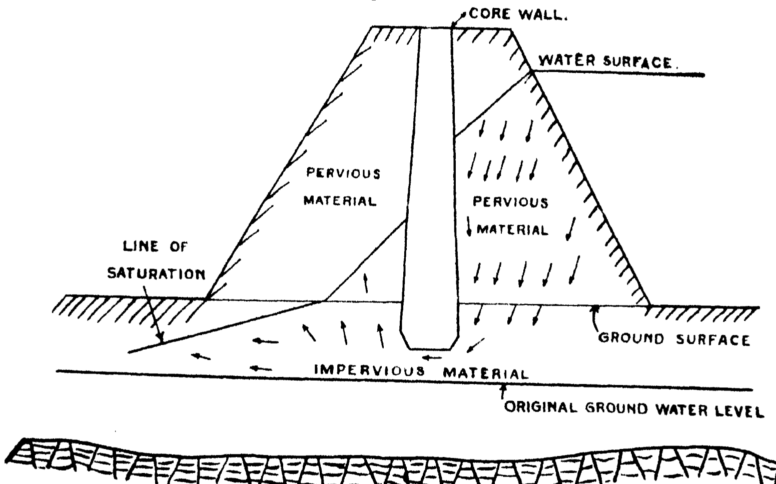
Fig. 21.





(*a*) In this case the material of the foundation and of the dam is pervious.

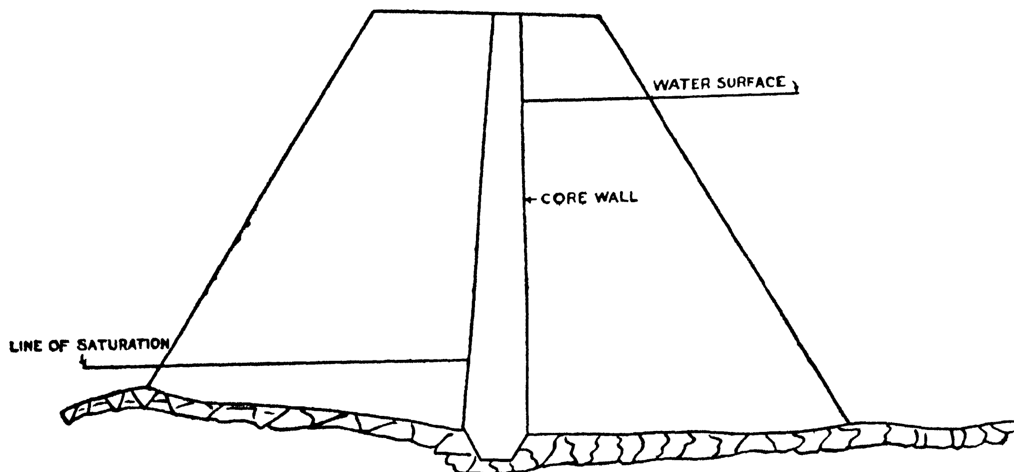
Fig. 22.



(*b*) If the core-wall is extended into the pervious foundation, the friction owing to resistance being greater, due to the introduction of the core into the foundation, the water rises to a lesser height on the downstream face and the line of saturation is lowered.

(*c*) In this case the foundation is made of impervious material and the dam of pervious material. The water from the upstream side passes down the core, as shown in Figure 22, to the downstream side. When it passes through the impervious material in the foundation and there heavy loss of head, due to friction, occurs on the downstream side.

Fig. 23.



(*d*) In this case, Fig. 23, the core-wall has been properly bonded to ledge rock due to which the line of saturation has been remarkably lowered.

Theoretically there should be no line of saturation and hence no flow of water on the downstream side. But the concrete or puddle core is never impervious and there are some seams in the rock, so there is some saturation on the downstream side.

The position of the core-wall:—The core-wall is frequently placed upstream instead of being directly on the centre line. Although the line of saturation in this case will have the same slope as in case (*a*) and (*b*), but it is lower.

Materials of the core-walls :—The core-wall is generally made of puddle or of concrete. The puddle is a mixture of sand. clay and gravel, which are mixed well in a pug-mill. The sand

and gravel give the necessary stability, and the clay the necessary cohesion. The most suitable mixture of these consists of :—

	Per cent.
Coarse gravel	60
Fine gravel	20
Sand	8
Clay	12
	100

The base width of the puddle core-wall is generally one-third the height, and top width one-sixth the height, battering uniformly on both sides. In many countries the puddle-wall is extensively used. Some countries use concrete-wall, which is comparatively thinner than puddle-wall for the same effectiveness.

Both puddle and concrete are impervious materials and offer a very great resistance to the flow of water through them. If the core-wall be embedded into the foundation rock and made up to the top of the dam, the dam is practically water-tight.

The width of the concrete core-walls are generally from 3 to 6 ft. at the top tapering uniformly with a batter of 20 on 1 on both the sides of it. Sometimes reinforced concrete core-walls are used, its width at the top being 1 to 3 ft. tapering uniformly with a batter of 20 to 1 up to the base. The walls are to be reinforced both horizontally and vertically with from 0·3 to 0·6 of 1 per cent. reinforcing steel placed near the faces.

The plane concrete core-walls must be provided with expansion joints, which should be coated with tar or similar paint to prevent adhesion of the concrete and possible crackings.

Reinforced concrete walls need not be provided with expansion joints as the reinforcement prevents large cracks.

Drainage of earth dams :— In order to lower the line of saturation the downstream part is drained artificially. The process is specially adopted for a dam made of impervious material. The drainage system is also necessary even if a core-wall be used. The foundation of a dam being made of impervious material and rock, the water that finds its way into the comparatively porous material cannot drain away properly through the soil of the foundation and consequently the downstream toe will be saturated with water when the downstream toe will be dangerously unstable.

Drainages are provided by digging trenches perpendicular to the axis of the dam and filling them afterwards with broken stones. Several laterals, consisting of 6 or 8 inches terror cotta sewer pipe, are laid to feed the main drain. The joints must be protected with gravel or crushed stone.

Criterion 3 :—Much care is to be taken for the upstream and the downstream faces, so that they may be stable under all conditions.

The upstream face is exposed to water. A slope not steeper than 1 in 2 with the horizontal, should generally be given to this face.

The slope of the downstream face is generally determined so that the line of saturation must fall within the base. Sometimes the slope is more flatter than the angle of repose of the material of that part of the dam.

The upstream and the downstream slopes that might otherwise be satisfactory may be unstable on earth dam having an unstable foundation. In that case both the upstream and the downstream faces are flattened in order to get an increased bearing area. The foundation soils are generally unstable when they are saturated.

Protection of top and downstream faces :—The surfaces of the earth dams must be protected from erosion due to rain and wind. For this purpose dense growth of vegetables on the surfaces will help the protection. For dams composed of impervious material, it is advisable to cover the surface with 10″ or 12″ of rich soil having good fertilizing property and then to be seeded to grass. In a high dam, the downstream face should be terraced so as to reduce the velocity of any water flowing down its slope.

The upstream face constantly wears away due to the impact of the waves. Hence, the upstream surface must be protected from wave action. It may be protected by the following ways :—

(*a*) Log booms may be placed in front of the slope and these will break waves before reaching the slope. Many earth dams have been protected by anchoring booms 3 ft. from the slope.

(*b*) Rip-rap is the best material to protect the upstream surface from wave action. Rip-rap is of two classes, randam and hand-placed. The randam rip-rap consists of stones dumped in place or tossed in place by hands. Hand-placed rip-rap consists of regular shapes of stones placed on edge on a prepared and graded bed, the minimum size of stones being used for this purpose being 15 by 15 by 3″ and the voids to be filled with smaller stones.

(*c*) Concrete lining on the upstream surface of the dam also protects it from wave action. But it must not be relied upon that concrete pavement keeps the dam water-tight. Sometimes concrete lining is reinforced about 0·3 per cent. of the effective area of the cross-section of the concrete.

The best practice is to make the concrete lining of square blocks 6 ft. by 6 ft. without any reinforcement. The blocks should not be less than 6-inch thick.

In order to allow the water in the embankment to drain away when the reservoir is drawn down quickly, it is essential to provide numerous weep-holes through the concrete.

Berms :—These are used on the dams to collect run-off. The rain water seeps fast into the coarse-sand dams made of less impervious material. In order to prevent surface erosion from the surface run-off, berms 8 to 20 ft. wide are generally used for dams above 25 ft. high. In order to prevent the rain water from flowing over the edge and down the slope the outer edges of the berms are higher than the inner ones.

Criterion 4 :—The dam must be water-tight and there must be no opportunity for the free passage of water from the upstream to the downstream face as water under pressure passing through it will dislodge the particles of the embankment.

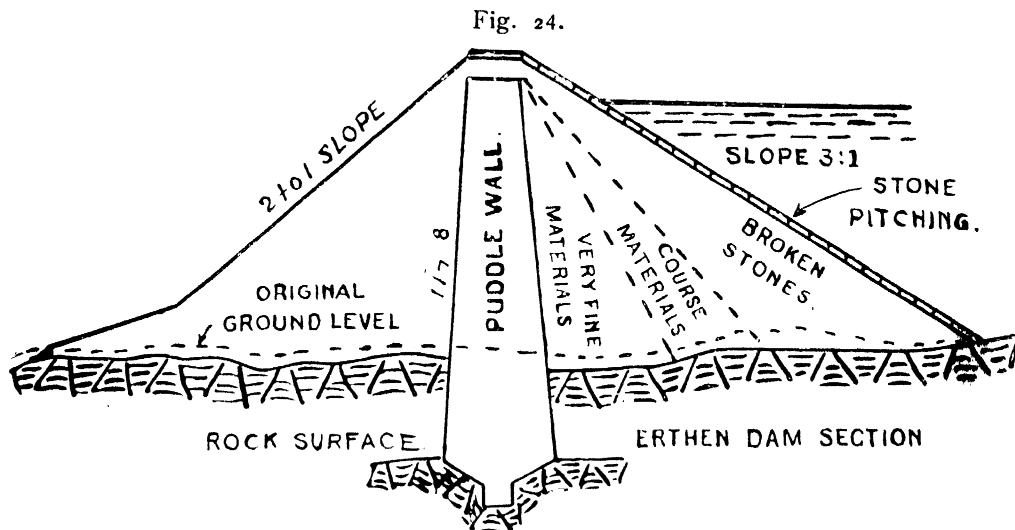
Boring animals such as muskrats may cause the free passage of water through the dam. If the core-wall be extended up to the high-water bed, there is no danger from such animals. Again if the upstream face be protected with rip-rap or with concrete slabs, such animals cannot injure the dam. The muskrat does not injure the downstream face generally.

The conduits through the dam should be placed below the original surface as it is very difficult to place a tight fill under the conduit. Otherwise water will flow around the conduit from one side to the other.

The careless placing of the materials may cause a blind drain passing from one side to the other one. So, coarse materials should not be placed in a continuous line from the upstream to the downstream face. The better arrangement of materials from the upstream face is coarsest materials next

coarse one than very fine material up to the core-wall in the middle as shown in the figure below :—

Fig. 24.



The upstream face is generally filled with coarse material.

The core-wall should be embedded up to the rock in the foundation and this will prevent the passage of water from one face to the other.

As there is a great chance of the water passing through the loose material, so the succeeding layers of the embankment must be compact and properly bonded. In order to lay the new surface over the older one, the latter must be moistened with water before placing the new materials over it.

Criterion 5 :—The earthen dam must be safe against piping action.

When water flows through a dam with a great velocity, it is capable of lifting any of the material of the foundation while it rises to the surface below the toe. The phenomenon is called *piping*. Due to piping there is undue waste of water.

In order to prevent piping action, the water flowing through or below the dam must have a very small velocity while it will be incapable of lifting any material of the foundation. According to Justin's opinion, the dam is safe from piping action if the actual velocity of flow under the dam does not exceed 0·5 ft. per minute for fine silt or coarser material.

According to Silchter's formula :—

$$v = \frac{hc}{lp}$$

where, v = velocity in ft. per minute of flowing water material.
c = transmission constant.
h = Actual head on the dam and which is the difference of the water-level on the two sides of the dam.
l = length in feet of the path of percolation.
p = porosity of the material in per cent.

p can be computed from the formula :—

$$p = \frac{100\ (w_1 - w_2)}{\frac{w_2}{S} + w_1 - w_2)}$$

where, w_1 = weight of the material saturated with water.
w_2 = weight of the material when dry.
S = specific gravity of the material.

The value of c, *i.e.*, the transmission constant, can be had from the following chart, which has been taken from the water supply paper No. 140, U. S. Geol. Survey :—

Some practical useful points in the preparation of site :—

(*a*) Stumps, trees and sod must be removed from the site of the dam and their roots must be grubbed out completely.

(*b*) There must not be any dividing plane between the dam and its foundation. For this purpose the surface of the completed foundation should be ploughed up and dragged longitudinally with barrow and then the embankment is placed upon it. It should then be moistened with water and rolled well when the first layer of the embankment will be forced down into the foundation leaving no dividing plane.

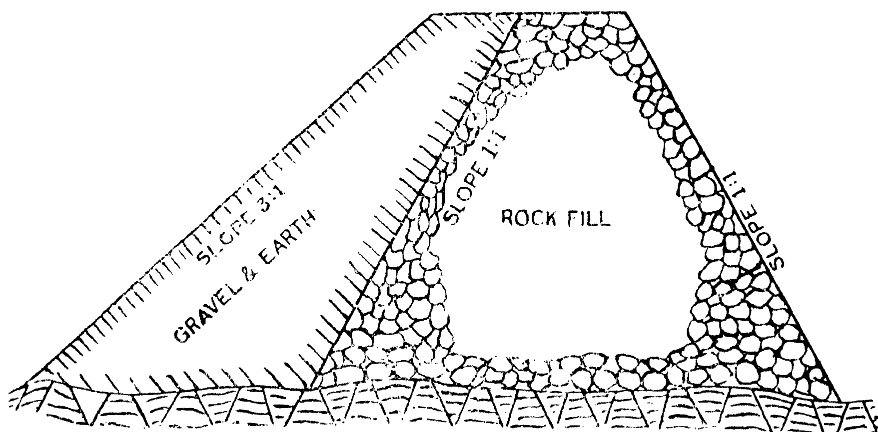
(*c*) Proper care should be taken so that the original surface will not be covered by the excavated material. For this purpose the foundation may be completed before excavating the trench for the core-wall.

Rock-fill Dams :—In a rock-fill dam the downstream part is filled with rock and the upstream part with sand, gravel and clay. In this type of dam less damage is caused by the overflow.

The rock-fill makes dam impervious and it is, therefore, evident that this type of dam is superior to ordinary earth dam. The wetted surface should be protected from erosion by rip-rap or concrete lining.

Rock-fill dams are used on uncertain foundations because the loose stones tumble into place when erosion takes place. They are generally given a fair width at the top with slopes varying between 1 in 1 and 2 in 1; they have the advantage of not being water-tight, although they can be made practically so, by placing on the upstream slope an impermeable layer of sand and clay. It is not advisable to allow the water to discharge over these dams, and like the earth dams they must be provided with sluice ways and waste gates.

Fig. 25.



Rock-fill Dam.

Hydraulic-Fill Dams :—The hydraulic-fill dam is the special type of an earth dam in the construction of which the materials are conducted to the dam through water, from a higher bank. The materials will then be conducted in flumes passing above the dam site and will be deposited in a semi-liquid state. The conditions most favourable to hydraulic-fill dams are borrow-pits at a sufficiently high level to sluice most of the materials by gravity into the dam

Water under pressure is discharged through nozzles against the bank of the borrow-pits with a high velocity so that it under-cuts the bank and breaks it up. The water carrying the loosened materials is then guided along the bottom of the pit to a sluice, consisting of a flume or pipe, through which it flows to the dam.

By this process a very compact and thoroughly mixed mass can be secured.

Timber Dam

This form of dam is recommended when the work is of temporary nature. In many cases it is used for the purpose of

diverting water. It is built in places where timber is plentiful and where the cost of transportation for cement, masonry, etc., is great. Rock-filled dams have been constructed successfully to a height of 70 feet. But this type of dam is not used in general for a height more than 20 feet.

The following are the disadvantages of the timber dam over the masonry one :—

(*a*) The maintenance charges for timber dams are large specially at sites where large floods are frequent.

(*b*) It leaks very often and the leaks are frequently so very great that it is very difficult to repair them.

(*c*) The timber dam can be built at a low cost, but its depreciation is more. The depreciation, interest and maintenance charge of a timber dam is greater than the interest charge of a masonry dam whose maintenance and depreciation charges are negligible.

In spite of these disadvantages the timber dam is built in many cases as it can be built at considerably less first cost than a concrete dam.

The timber dam may be divided into three classes :—

(1) The crib dam.
(2) A-frame type.
(3) The beaver type.

(1) *Crib Dam* :—These are usually used for temporary work and may be used on rock foundation or soft ground.

The crib dam consists of cribs of round or squared timbers drift-belted together and topped by a blank deck. These are used for low heads up to 30 feet where timber is cheap. In most cases it is filled with rock-fragments or boulders. The usual spacing of the timbers are 8 feet centres both ways. If it be not filled with stones, the upstream slope must be very long in order that it may be stable.

The bottom timbers of the cribs must be pinned to the foundation with sheet pilings and a plank decking is provided to prevent leakage.

The foundation should be on rock. The upstream face of the timber dam is generally tapered, and a portion of it up to 4 feet is made vertical if sheet piling is to be used.

For low dams on soft foundations of crib type, the upstream face is long and sloping and frequently stepped in order to drop the water without great disturbance.

A heap of stone, gravel or fill of earth and sand is to be deposited in front of the upstream face.

A-frame Type :—The A-frame type of dam may be called, in other words, the rafter and strut framed dam, as it consists of rafters and struts framed in the form of the letter 'A'. It is built of squared timbers and planks where the former is used as struts.

The upstream face of it is a flat-deck made of reinforced concrete sloping to a maximum angle of 30° with the horizontal. The foundation is on ledge rock. At first the sills are fastened to the ledge rock by wedge bolts preferably grouted in. The struts are then framed to the sills and held in place by cross-bracings. The upper ends of the struts are framed to the walls when the entire structure is thoroughly driftpinned together so as to form bents. These bents are generally spaced 8 to 12 ft apart. The studs are placed across the bents, upon which the flat-deck is placed. The studs must be nailed to the lagging.

Mortar, concrete, or earth-fill is to be provided at the bottom of the deck in order to prevent it from slipping downwards.

In order to see that the dam of this type may not slide, the struts are to be placed being inclined towards the upstream side.

The Beaver Type :—This type of timber dam is only used for very low-heads. Here round timbers are used for bents. Between the round timbers are spaced spacer logs being drift-pinned to the others, the intermediate spaces being filled with rocks. The deck is of plank.

A fill of earth or sand is to be deposited at the upstream end.

Spill-way Timber Dams

For spill-way timber dams the crib type is generally used and is advantageous. It must be protected against erosion from overflow if the foundations are soft. For this reason aprons are provided to the stepped downstream face in order to protect the foundation.

A sheet piling is to be used on the upstream face and for this reason a vertical upstream face of 4 feet is to be provided in order to afford better opportunity for fastening it to the dam.

Comparison :—Of the three types of timber dam, A-frame type is most economical as it requires less timber. But there is the great danger of its failure if it be neglected. Hence good workmanship must be required at the time of building this type of timber dam. No sheet piling is to be used for this type of dam.

The beaver type can be used only for very low-heads and where timber is very cheap.

The crib type can partly be constructed on land, floated into place and sunk by filling with stone.

Sheet piling must be used for this type on both the faces. The other types cannot easily be constructed in deep water except this one.

For spill-way timber dams, crib type can only be used.

Foundation of Dams :—The character of the foundation on which they rest is the most important factor in the stability and permanency of dams. The most important function of the foundation is to prevent the passage of water below the structure and to afford regidity of position to the super-structure. The design of the foundation depends on the character of the material at its site, as to hardness, strength and porosity, the height and the weight of the super-structure and the effect of its over-flow.

The foundation sites are in rock or alluvial formation.

In rock its hardness stratification, condition and shape of the surface determine the foundation. The rocks are in general divided into the following classes :—

(*a*) *Primitive rock* :—Granite, Sienite.

(*b*) *Metamorphic rocks* :—Gneiss, Sienite gneiss, green stone, trap and basalt.

(*c*) *Secondary rocks* :—Sand-stone and soap-stone.

(*d*) *Tertiary rocks* :—Lime-stone and slate.

Physical Characteristics of Rocks

Rock.	Weight per cu ft.	Crushing strength per sq. ft	Remarks.
	lbs.	Tons.	
Granite ...	180	750	Unstratified.
Sienite ...	" ...	" ...	"
Gneiss ...	" ...	700 ...	"
Sienite-gneiss ...	" ...	"	"
Trap ...	" ...	" ...	"
Basalt ...	" ...	" ...	"
Green stone	" ...	" ...	"
Sand stone ...	150 ...	609 ...	Stratified.
Marble ...	168 ...	500 ...	Unstratified.
Lime stone (hard) ...	" ...	" ...	"
Slate ...	175	600 ...	Stratified.

Having examined the above properties of rocks, the design of the foundation will be based on the following points :—

(a) *In hard unstratified rock* with level surface no foundation is necessary. The homogeneous ledge will not permit water to pass beneath its surface. It will safely carry the super-structure which is anchored and keyed to it.

(b) *In hard but stratified rock* a cut-off wall must be constructed along the upstream side of the spill-way structure in a trench excavated from the rock to a sufficient depth to pass below these strata which are less than 2 ft. thick, its upper portion becoming part of the super-structure. The rock ledge may be of sufficient solidity to carry safely the spill-way.

(c) *In soft but stratified rock* the cut-off wall is necessary. The soft disintegrated upper portions are completely removed and then the super-structures are to be founded on the rock surface. But an apron must be constructed on the downstream side of the super-structure to receive the over-fall and resist its erosive force.

Hence, there is no particular difficulty in securing a reliable foundation where the stream bed is of *granite*. Occasionally, it will be found that what appears to be solid bed rock is merely a boulder underlaid with mud or earth. For this reason drill holes should be made into the rock from 6 to 12 ft. in depth and 2 inch in diameter in order to be sure that no defects exist.

In order to prepare a foundation on the granite remove all the loose rock and blast out a shallow trench to receive the bottom of the dam.

Wherever the stream bed is of *lime-stone*, it is necessary to make a number of drill holes over the surface on which the dam will rest. These holes should be spaced at intervals of 12 ft. and blown thoroughly with compressed air at a pressure 4 to 6 times greater than the atmospheric pressure. It will be found that air entering one hole will blow out of the adjacent holes, showing a fissure or cavity with which the holes connect. In order to close up all the small cavities and seams, grout must be forced into the holes with grouting machines until all of the fissures and openings in the underlying rock are thoroughly filled and the whole mass solidified. The common mixturing of grout contains 50% cement and 50% sand with sufficient water to make a liquid that will flow freely.

Dams built on lime stone must have adequate cut-off wall at the upstream edge, the bottom of the wall should be well below the permeable strata. In case of hollow reinforced concrete dams, the grouting in the cut-off wall trench, and in the trenches which receive the buttresses will be sufficient.

Foundation on Alluvials :—The alluvials consist of gravel, sand clay, loam, marl and peat.

Gravel is the fragments of rock reduced to pebbles by the atmosphere and water.

Loam is a mixture of clay and sand.

Silt is a fine earthy sediment.

Marl is a consolidated mixture of clay and carbonate of lime which readily disintegrates where exposed to atmosphere.

Peat is decomposed vegetable matter containing much water near the surface.

Physical Characteristics of Alluvials.

Material.	Bearing power per sq. ft	Angle of repose.	Coefficient of friction.
	Tons.		
Gravel	2-3	38°	·78
Sand dry & loose	,,	28°	·53
Sand wet	,,	,,	,,
Clay dry	4-6	45°	...
Clay damp	,,	,,	1·0
Clay wet	,,	15°	·27
Loam dry loose	,,	35°	·7
Loam wet	,,	35°	,,
Mud	,,	0°	,,
Gravel & loam	2-3	38°	·78
Gravel & sand dry	8-10	45°	1·0

Having examined the properties of the alluvials the design of the foundation will be based on the following points :—

(*a*) In compact alluvial gravel and sand with no interior sand strata, a cut-off wall is required to a depth below the river bed equal to one-fourth of the maximum water head. It may be placed directly upon the levelled and cleaned hard surface if the aggregate weight of the super-structure and water does not exceed 2 tons per square foot ; and if more than 2 tons per sq. foot bedding piles must

be driven to support safely the super-structure which may be placed directly upon them, or a concrete foundation floor may be laid, the pile tops being imbedded in it. An apron is necessary on the downstream side.

(*b*) In soft alluvials of clay, gravel, sand loam, silt or peat upstream cut-off walls are required to a depth penetrating into impermeable material or to rock with a pile driving foundation having aprons on both upstream and downstream side.

(*c*) In clay with no sand strata for a depth of one-fourth of the maximnm water head, the upstream cut-off wall must be constructed to a depth of one-third of the maximum water head, and these must form a part of the foundation floor, which rests upon the bedding piles and is extended up and downstream of the super-structure base as aprons.

Sand Foundations:—Keobig states that: "No material other than solid bed rock offers a more secure support than sand which is properly confined; therefore to make it serve as a safe foundation for a permanent dam is entirely a problem of making adequate provision against the only disturbing element—a current of flowing water." Dams may be built successfully on sand foundations if the water may be prevented from flowing through the body of the sand and the stream bed from being washed away. If anyhow water can percolate through it, its velocity will increase gradually and in short time the whole structure would be destroyed. To remedy this the path of percolation must be of sufficient length when the resistance to flow will be great enough to retard the flow of water through the sand. The provision which is needed against the current of flowing water comprises one or more cut-off walls, either of wood or of concrete, of which one should be at the upstream edge and one at a considerable distance downstream. The depth of the cut-off walls below the surface of the stream bed must not be less than 2 times the height of the dam for medium heads up to 15 feet.

Coffer dam:—During the construction of a power dam the water must be excluded from the construction site. Hence a coffer dam is to be constructed at first. The phenomenon is called the *coffering*, which is the first operation preparatory to the construction of any part of the dam.

The coffer dam consists of two parallel rows of sheet piling the intervening space between them being filled in with puddle.

Sometimes three or even four rows with a puddle wall between each pair have been adopted in order to give the greater stability and make the dam more water-tight.

It requires the judgment, born of experience, to confine the means employed for erecting such a dam of temporary character with economic cost. The failure of a coffer dam will damage the permanent work under erection.

The puddle wall should not be more than 5 feet in thickness. Bolts through coffer dams should be avoided as far as possible as the puddle in setting leaves vacant spaces between them causing leaks ; to remedy this, iron discs through which they pass have been used ; these discs are embedded midway in the puddle wall.

The good puddle for this purpose is made by mixing clay with one-fifth of its weight of water and the mixture must be highly plastic paste. The usual method of preparing puddle is to mix the local clay with sufficient water to form a soft paste, a pug-well being used for the mixing. The clay having a sandy loam containing 50 % sand is incapable of producing a mass impermeable to water. On the other hand, if clay be rich in colloidal matters, it will have an abnormally high shrinkage on drying ; the abnormal shrinkage is objectionable as puddle, which shrinks greatly, usually cracks, and becomes unreliable. The clay, being rich in colloidal matters, should be mixed with fine sand so that the shrinkage after drying must not exceed one inch per linear foot.

The two sides of the sheet pile curtains are to be covered with rip-rap facing fill where rip-rap is loose rock thrown up against the curtains to break the force of flowing water or resist pressure.

The coffer dam with sheet piling can be constructed when the river-bed is of clay and sand. It cannot be constructed on rock-bed.

In rock-bed and shallow water where the velocity does not exceed 3 feet per second and where the head is low, a dike serves the purpose of a coffer-dam.

The dike is a thrown-up rock and earth bank. It consists of a core of loose rock of all sizes and an earth or clay and sand facing fill.

In rapids a break-water, *i.e.*, a log-crib is to be constructed.

In rock-bed having high head of water A-type timber dam is to be made with struts only.

Headwater Control and Dam Accessories

Headwater Control :—It is naturally very desirable to maintain a constant level of water above the dam. The water surface fluctuates to a considerable extent during different seasons of the year due to which the following disadvantages arise :—

(1) If the water-level falls below the normal, the working-head of water is thereby reduced and thus less water power than the desired value is available.

(2) If the water-level rises above the normal, specially during the flood, such lands which have not been included in the flowage area will be over-flooded. Hence, the elevation to which headwater may be allowed to rise, is limited by land or water rights which are not owned.

In order to discharge the flood-waters, the spill-ways should be as long as practicable in consistence with economy, to limit the head on the crest. But in many cases sufficient length of the simple crest cannot be obtained.

Hence, a mechanical process adopted to maintain a constant level, both at times of low-water and floods, is to provide FLASH-BOARDS, upon the top of the dam, which are arranged to be raised or lowered with the variation of the water-level.

Of the numerous designs of the flash-boards the followings are the most common types :—

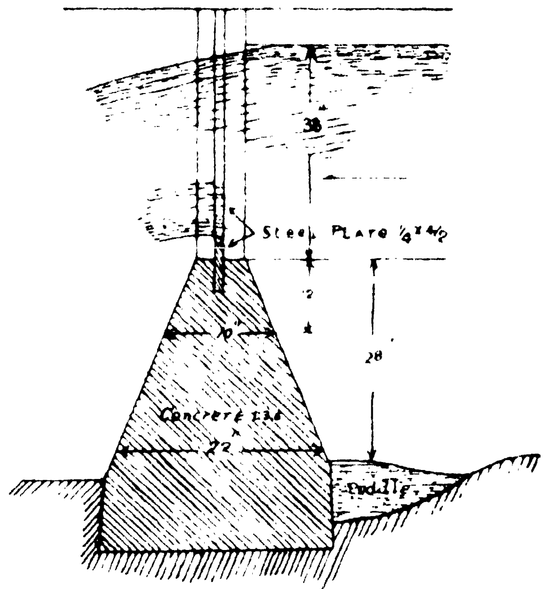
(1) Stationary flash-boards.
(2) Sliding gates.
(3) Tilting gates.
(4) Tainter gates.
(5) Drum gates.

Flash-boards are wooden boards or panels mounted on the crest by which the upper pool level is temporarily raised for the purpose of accumulating an increased volume of water during a non-operating period.

Use :—In this manner the upper pool by pondage is utilized as a storage reservoir, principally during the low-flow periods, by placing some movable addition upon the spill-way crest, arresting and holding the natural flow in the upper pool, no water being allowed to overflow the spill-way or pass through the turbines during a certain period, generally some portions of the night, and then using the natural flow *plus* the accumulated volume, during

the operating period. When this proceeding is feasible,—which is not always, nor generally, the case, because it interferes with and disturbs the natural conditions of the flow in the stream and thereby is likely to interfere with the rights and ownership, in and to the water, of others—and when the operating period can be confined to ten hours, the power output of the continuous flow can, in this manner, be practically doubled.

Fig. 26.



They may be held in place by permanent steel supports embedded in the concrete or the support may consist of iron rods designed to bend and release the boards when the height of the water reaches a certain limit. Flash-boards are now seldom used except in minor installations.

Stationary Flash-Boards:—This type of flash-boards consists of a row of wooden panels or boards placed on the top of the crest of the dam and are supported by iron pins or steel pipes. These iron pins or pipes are set vertically in holes or sockets previously provided in the concrete structure. The iron pins are designed to bend over and loosen the flash-boards when the water surface in the pond reaches a certain elevation. The boards or panels are fastened loosely to the supports and are lost when the supports bend over, unless they are removed before anticipated high water. See Fig. 20, page 89.

Guard-pins are provided at intervals into the sockets in the crest of the dam in order to facilitate the handling of a barge for removing and restoring the flash-boards The ends of different sections or boards overlap each other and thus a fairly water-tight joint is provided. The boards usually have unplaned edges. Ashes or similar chalking materials are used to make them tight.

For sealing the joint between the lower edge of the boards and the masonry, a composition of cinders and straw, being

well mixed together, will be a good application with a satisfactory result.

In many cases it is possible to remove the flash-boards in advance of floods. In such cases they are made in panels and provided with handles.

For wide streams the usual method of handling them is by means of a scow provided with steam-driven derrick, while for narrower streams specially designed cableways with chain hoist have been used with very great success.

In order to design the steel pins which are to be bent when water rises to a certain level above the crest, proceed as follows:—

Let H = height of water in feet above the dam crest when pins begin to bend.
h = height of the flash-board in feet.
s = spacing of the pins in feet.
d = diameter of the pin in inches.
f = extreme fibre stress of the pin in lbs. per square inch.

The pressure on the elementary area δx

$$= 62{\cdot}4\ s\ (H - x)\ \delta x \text{ lbs.}$$

Its moment about A = $62{\cdot}4 s\ (H-x)\ x\ \delta x$ ft. lbs.

$\therefore$ the sum of moments of all the pressures upon the pin AB about A.—

$$= \int_0^h 62{\cdot}4\ s\ (H-x)\ x\ dx$$

$$= 62{\cdot}4\ s \left[\frac{Hh^2}{2} - \frac{h^3}{3}\right] \text{ foot lbs.}$$

From the law of Bending moments :—

$$\frac{M}{\frac{\pi d^4}{64}} = \frac{f}{\frac{d}{2}}$$

or, $M = \frac{\pi}{32} d^3 f$, where M = Bending moment in lb-inches.

$$\therefore 62{\cdot}4\ s \left[\frac{Hh^2}{2} - \frac{h^3}{3}\right] \times 12 = \frac{\pi}{32} d^3 f$$

$$\text{or, } H = \frac{2\pi d^3 f}{32 \times 12 \times 62{\cdot}4 \times s \times h^2} + \tfrac{2}{3} h$$

$$\text{or, } H = \frac{d^3 f}{3814\, s\, h^2} + \frac{2}{3} h \quad \ldots \quad \ldots \quad \ldots \quad (A)$$

Tests on various steel pins show that their failure will occur for a fibre stress lying between 42,000 to 58,000 lbs. per sq. inch.

Taking f = 50,000 lbs per sq. inch which being the mean value.

The equation (A) reduces to : –

$$H = \frac{50{,}000\ d^3}{3814\ s\ h^2} + \frac{2}{3} h$$

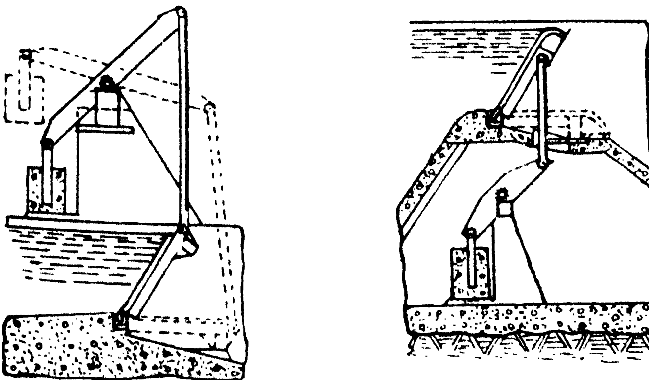
$$\text{or, } H = \frac{13{\cdot}1\ d^3}{s\ h^2} + \frac{2}{3} h \qquad \ldots \qquad \ldots \qquad \ldots \quad (B)$$

Tainter Gates:—These form of gates are very largely used owing to their ease of operation. It is built of steel throughout in the form of a sector.

The gate is pivoted at the centre and its outer end is lifted by a chain or cable from the track above by a permanent winch. A portable hoist also can serve the purpose of a winch. In order to make it water-tight, the bottom of the gate is fitted with a sill block of timber which bears against the edge of a steel plate let into the crest of the dam.

Tilting Gates:—This type of flood gate consists of a flash-board at its lower edge to the crest of the dam, the upper end of it being connected by two steel cables to heavy concrete roller counter-weights where the cables are wound in grooves around each end of the roller. The flash-gate is capable of moving from a more or less vertical to horizontal position.

Fig. 27.



The Stauwerke Tilting Gage

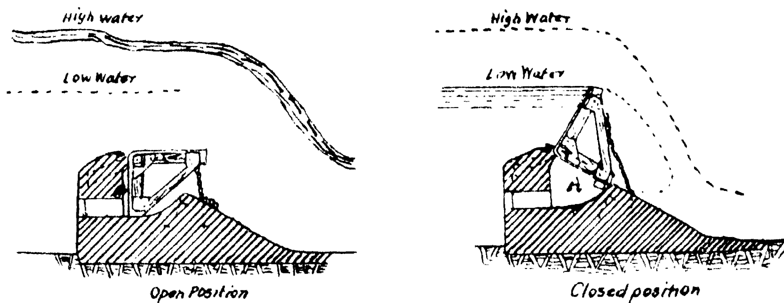
The counter-weight rollers travel on inclined tracks, each end being provided with a geared drum which engages a rack to prevent slipping.

The flash-board maintains its upright position when the water-level above the dam is within the normal level. If the water rises above the normal level, the

additional pressure will cause it to tilt over further so long as it is not horizontal. As the water subsides, the gate will automatically rise until the normal water level in the pond is reached.

Drum Gates :—This type of gate has got two leaves, one above the other. Both the leaves are under water pressure. The upper leaf is longer than the lower one.

Fig. 28.



Stickney Type of Drum Gate

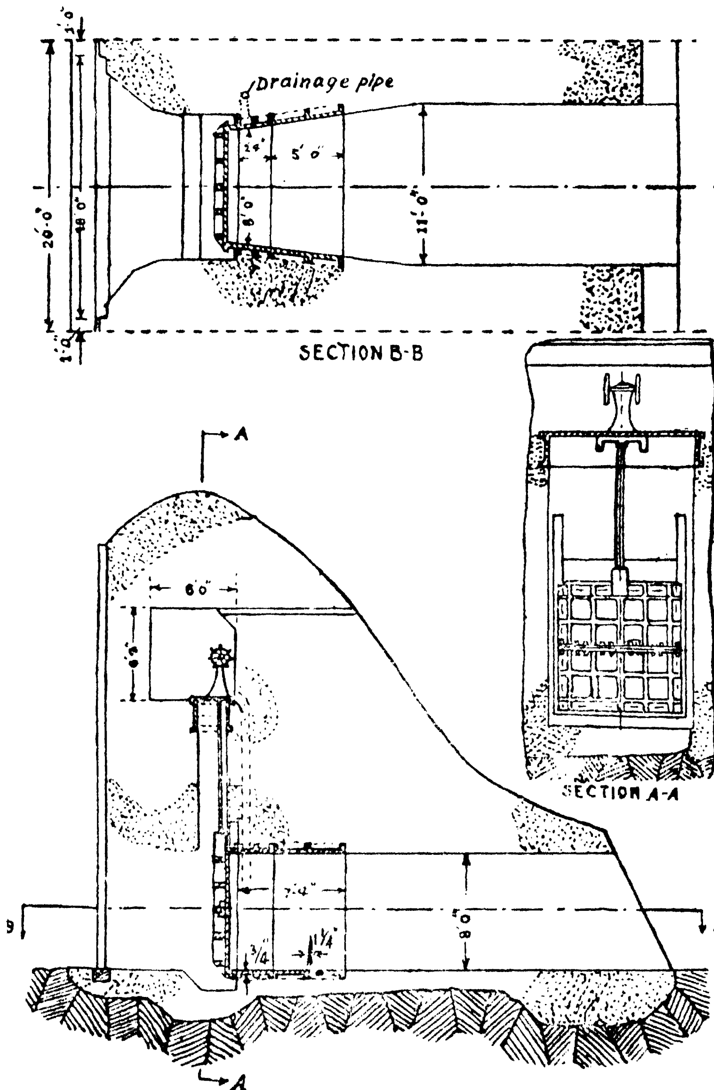
During low water, the constant water pressure upon the lower leaf holds the gate as the water is admitted into the compartment A. During high water due to floods the pressure upon the upper leaf is greater than that upon the lower one. Consequently the greater pressure forces the gate to its lower or open position. When the flood is discharged, the water surface drops to its normal position and the gate automatically rises to its normal position. The gate must be anchored with the crest of the dam with a chain.

Sluice Gates :—Sluices are placed in the lower part of the dam to assist the spill-way in the discharge of the floods. In concrete dams they are usually installed to serve the following purposes :—

(*i*) to be used as outlets for storage reservoir.
(*ii*) to empty the small reservoirs for repair.
(*iii*) to drain flood waters, if necessary, without operating the flash gate.

The conduit must be designed in such a way that a partial vacuum and eddies can, on no account, be formed, and these are very conducive to erosion. Sudden change of section must be avoided. A change of section is absolutely necessary at the control, and for this, the area of the sluice should be reduced gradually between the entrance and the control and after the control it should again be increased gradually.

Fig. 29.



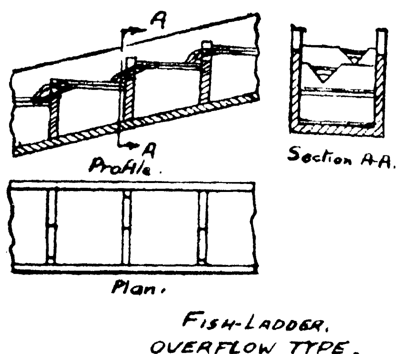
Rectangular sluice gates and valves having both sliding and roller bearings are generally used. Below the control the concrete must be protected with a cast-iron or steel lining which is to be anchored well and provided with ample holes to prevent rupture from seepage water. The entrance to sluice gate is to be protected by heavy rocks. For high-head dams the control is generally placed at the outlet. But for dams of medium size the control is placed inside the dam when much care is to be taken for motors, geared hoists to keep them in good condition from dampness.

Fish Ladders :—The natural tendency of the fishes is to swim against the water current. Hence the state law may demand that the dam is to be provided with means whereby fishes can easily ascend or descend safely according to their natural habits, in search of food. A fish ladder is an inclined trough in which the water flows from the upper to the lower pool, with a velocity

against which fishes can easily swim. Of the various types of ladders one type is shown here.

This is the over-flow type where each baffle board contains a V-shaped notch over which the water spills. The fall is low enough for the fishes to jump.

Fig. 30.



FISH-LADDER, OVERFLOW TYPE.

The second type is a sluice type. Here each baffle board contains an opening through which the water passes. The slope of the ladder is to be designed for such velocities of water as allowed by the state-laws.

The trash-racks :—These racks should be so constructed as to give sufficient area for passing the desired quantity of water without excessive loss in head. Allowance should be made in the accumulation of trash as a factor in the restriction of water passage.

This may be placed upstream of these to intercept all small floatage. A trash-rack is sometimes arranged at the throat of the forebay, though one may be provided for at the head gates.

Full-load velocities at the racks are not usually affected by head losses, because the velocity is generally limited to that at which racks can be easily used. Higher velocities can be used where mechanical racks are installed than for hand operation. Velocities, at full load in the gross area of racks, between 1·75 and 2·0 ft. per second are usually adopted, although slightly higher and lower velocities have been used.

Needles are stop logs placed vertically side by side, footing against a sill secured in the sluice bed, and supported on the top by horizontal strain members or the platform. They are not as ready for manipulation as are stop logs of the horizontal type and it is also more difficult to make them water-tight.

They are removed by lifting each from its seat and hauling it out bodily. Each should be provided with a hole near the top through which to pass an anchor rope, to hold the needle in case it gives way when being handled. They are invariably not frequently used.

Stop logs :—There is a simple, effective and inexpensive device for head gate service, being placed and operated in the same manner as when employed for the closing of overflow sluices. They are the simplest forms of crest gates and are removed one by one as the need for increased discharge occurs.

Use :—They are installed and removed by the hand and should not be too large.

Construction :—They are square timbers placed horizontally and guided into place by means of a vertical slot at each end of the opening above the gates—one over another in the sluice span, their ends resting against shoulders arranged in the supports; they are operated by hand or mechanical devices from the operating platform and can readily be formed into a water-tight curtain. They are held in place by water pressure.

Disadvantage :—The difficulty of installing and removing them.

Dams at Tata, Bombay

The lakes are formed by constructing along the three valleys, dams with coursed rubble masonry faces and with beat of uncoursed rubble masonry.

There are two dams for Lonawala lake, one a main dam 3600 ft. long across the valley and a smaller dam 1900 ft. long in which the sluices are built, the former being 40 ft. high to the bottom of the foundation and 26 ft. above the lowest level of the valley across the headwaters of the Indrayani river. The dam has ample waste escapes, scouring sluices and outlet sluices and the top carries a roadway with parapet on each side.

Walwhan dam is 4500 ft. long and 75 ft. high to the bottom of the foundation, of 68 ft. above the lowest level of the valley. The dam is provided with waste escapes and ample sluices of large dimensions and the top carries a footpath with parapet on each side.

Sirwata dam is 8,000 ft. long and 100 ft. high to the bottom of the foundations and 92 ft. above the lowest level of the valley.

The dam is provided with a waste weir 2470 ft. long and the top carries a foot-path with parapets on each side

The dams are designed so that the resultant of the water pressure and the dam weight at any height cuts the base at the height within the middle-third of the thickness. The weight of masonry being taken at 156 lbs. per cu. ft.; and at no other point either with the lake full or empty is the pressure on either edge to exceed 6 tons per sq. foot. The total quantity of rubble masonry in the 3 dams is 926,000 cu. yds.

The outlet of Shirwata lake is connected to Walwhan lake by a tunnel 5,000 ft. long through the hills the formation being hard trap rock. At the inlet of this tunnel there is a head wall containing ample sluices to allow Shirwata lake to be discharged into Walwhan lake as desired to meet the required conditions of draw-off.

In the dams and head walls the sluices are of '*stony*' *type* in which the pressure against the sluices is transferred to the fixed work through systems of free rollers which greatly reduce the friction compared with simple sliding gates so that the weight of the sluice itself is ample for closing purposes under the worst conditions. The sluices are built up of wrought steel plates and sections, provision being made to reduce the leakage to the required amount by an adjustable stanchion bar at the top of the gate which shuts into the lintel simultaneously with the sluice skin shutting on the sill, and two adjustable side bars which can be adjusted when the sluice is closed from the downstream side. The grooves, lintel and sill are of C. I. built into the masonry of the dam, the grooves being provided with sluice plates for protecting the roller from the force of the water, an important point as regards the durability of rollers and roller plate.

To reduce the labour of lifting the sluices, *concrete counter blocks* are provided suspended on pitch chains passing round sprocket wheels driven by spur gearing. The ease of the operation is such that one man operating the lifting crab of the 6 ft. and 7 ft. sluices can open or close them at the rate of 1·75 ft. per minute.

There are in all 13 sluices,—four 5 ft. × 5 ft. for operating against a head of 74 ft.; five 6 ft. × 7 ft. for operating against a head of 52 ft.; and four 6 ft. × 7 ft. for operating against a head of 25 ft. Sluice gear houses are provided built of coursed rubble masonry rough dressed with powder of brick in Portland cement or mortar and rendered with portland cement inside and out.

For scouring purposes another type of sluice is used having frames and doors of C. I. with gunmetal faces pinned on and scraped to a water-tight bearing, the doors being wedged tight by means of metal cast in the back and wedges on guide bars bolted on sluice frames. They are operated by wrought iron rods fitted with bevel and worm gearing, the latter being used for starting and closing the switches, and the bevel gearing for opening them. The whole of the gearing is enclosed in C. I. casing with a hood on the top, an index plate with a pointer showing whether the sluices are open or shut.

There are in all 7 sluices, two 5 ft. × 5 ft. operating against heads of 26 ft. and 28 ft., respectively ; one 3 ft. × 3 ft. operating against a head of 61 ft. ; and four 3 ft. × 2 ft. operating against a head of 12 ft.

The masonry work throughout is built with lime mortar. Two varieties of lime are used, Kankar and Bagalkoti. The proportion of the lime is varied to suit the materials available. Daily tests were made of all mortar used. The tests were of two kinds, one for hydraulic properties and the other for strength. The most important test being for compressive strength which was made in a hydraulic compression testing machine.

The design of the dam provides for a maximum stress of 6 tons per sq. ft.

Dam and Storage Works

The dam is situated at the junction of Nilla and Mulla Rivers at Mulshi village in the Poona District. The rainfall on the catchment area ranges from 150-250 inches a year.

The design is based on a weight of masonry of 140 lbs./cu. ft., which gives a maximum stress, with the reservoir full of 9·1 tons per sq. ft., no allowance being made for uplift. The average weight of stone actually used is 173·4 lbs. and that of the dry mortar 121·5 lbs. giving an average weight of masonry 148·5 lbs./cu. ft. The dam contains approximately 22 million cubic feet of masonry, being one of the largest in the world. The total length of the dam is 5,100 ft., including 1,500 ft. of waste weir, and the maximum height 160 ft., including top works. Deccan trap rock from the foundations or from near-by quarries was used with lime mortar composed of 1 part of fat lime, 1 part of surkhi and 3 of sand. The lime stone was brought at Chinchwad, burnt with cinders or coal in open kilns and sent to the site ready slaked and screened. Surkhi was made at the site from ordinary clay soil, made into bricks by hand and burnt in clamps with wood fuel. Crushed rock screened through mesh screens was used for sand, as practically no river sand was available near-by. The minimum crushing strength specified for mortar was 150 lbs./sq. in.

Before the construction of the plant was complete, ordinary rubble masonry in lime mortar with kalli facing was built ; when tracks from the quarries and round the dam were laid out, cyclopean masonry in lime mortar was adopted and steam cranes were used for handling pumps and concrete. This kind of work was very slow and was finally abandoned after 618,000 cu. ft.

had been done. The failure of this system, which had advantage from the point of view of the somewhat lower proportion of mortar required, and the increased weight of masonry obtained, was mainly due to two causes :—

(*a*) The local labour was unused to the handling of 2 or 3 ton blocks of stone, and it was difficult to train fresh crane group every season, which was necessary owing to the floating supply of labour. In fact if the progress, later obtained with ordinary rubble masonry, had been required with cyclopean work, the cost of the special plant (chiefly cranes and heavy steel framed wagons) would have had to be almost trebled.

(*b*) The steam cranes did not have a caterpillar track, but ran on 5′ 6″ gauge rails, so that considerable expense and inconvenience were caused in changing at the various working levels.

When the use of cranes was discontinued and hoisting towers were available, lime concrete consisting of 1 part of mortar and 2 parts of metal was placed between the two face walls of rubble masonry each about 8 ft. thick on an average. Instead of the plums, stones as big as could be conveniently handled were laid in the concrete. A total of 41,98,000 cu. ft. of work was done in this way. When higher levels were reached, however, and as the working space became more restricted, the concrete was omitted and simple rubble masonry was adopted.

The faces of the dam are of kallis prepared from selected stones, not less than 6″ sq. in face and 12 inches to 18 inches long. *Headers* were placed at the required intervals. The joints were raked out to a depth of 3 inches and the upstream side is pointed with a mixture of 1 part of cement and 2 of sand by volume, with an addition of *Ironite*.

Towards the south flank of the dam, as rock was not met within a reasonable distance, a trench ranging from 10 to 14 ft. in width was cut into the side of the hill, till solid rock was exposed. The trench was filled with 1 : 2 : 4 Portland cement concrete, the junction of this cone wall and the dam is protected on both sides by bank of selected earth well rammed and consolidated, the upstream bank faced with rubble pitching.

The top work consists of a simple roadway 15 ft. wide over the bulk head section only, as apart from the spill-way the roadway is paved with kallis and pointed with cement, but it is not accessible to the public.

The spill-way, which is of masonry in cement mortar, is of gravity type designed to discharge 68,000 cu. secs. with a height of 6 ft. of water. Provision has been made to allow automatic gates 40 ft. in span, to be installed later, should they be considered necessary. Spill-way discharge tunnel ranging from 30 to 220 ft. in width was cut in the hill to lead the water to the main river. Although this involved an excavation of 11,600 cu.ft. of rock, only a small part of the stone was suitable for building the main dam.

All excavation for the main dam and most of the excavation for the weir channel was done by hand. The foundation throughout was taken down at least 3 ft. into unfissured hard rock. As further precaution, 3-inch core holes were driven to a depth of not less than 15 ft. at every 100 ft. or less, as was found necessary to ensure that the layer of rock met with was of suitable thickness and nature. While no gallaries have been provided, there are drains and peepholes at every 50 ft. in the lower 100 ft. of the dam.

Although the plant installed would not be considered extraordinary from similar works in Europe or U. S. A., the layout of the plan was on an unprecedented scale for India. All metal and sand were obtained by crushing stone. The 28 mortar pans were 9 ft. in diameter underdriven and worked in batteries of 6. Each gave an average output of 36 cu. ft.

The hoisting of all concrete was done after the work came above ground level by 8 Ransome shooting towers, spaced roughly 400 ft. apart and each 210 ft. high. The shoots had to be adjusted to a slope of about 40° to the horizontal to allow the flow of lime of proper consistency. When the concreting was stopped, the towers were used for hoisting mortar. The shoots were renewed and the towers heightened each season. After the dam reached an average height 20 ft. above ground level, 8 locally-made rubber hoists were erected between the towers.

Pneumatic drills and hammers were used in the quarries and on the excavation for the wire channel as soon as power was available.

All plants except portable and the locomotives were electrically driven. Power was obtained from a 1,500 K.V.A., 22,000/2,200-volt, 50-cycle substation from which 2,200-volt lines radiated to the quarries and various sections. All machines taking 50 h.p. and upwards were driven by standard textile mill 2,200-volt Induction Motors. All the rest were driven by 440-volt motors from suitable banks of three 50 or 15 K.V.A. single-phase 2,200/220-volt transformers from which a 3-phase, 4-wire system

ran for power and lighting. A total of 1,500 h.p. was thus being used.

The site of the works was connected by 24 miles of 2′-6″ gauge of construction railway to the G. I. P. main line to Poona at Chinchward. The railway apart from bringing plant and daily stores to the site was used for the transport of lime and during the part of the year an average of 4,000 cu. ft. of slaked lime was transferred daily to the site in box wagons.

At the time of maximum output a labour force of 10,000 was engaged.

The actual construction was completed in 5 working seasons, the maximum output being obtained in the second season with a record of 9,98,900 cu. ft. in the month of April, 1925. Masonry building was entirely stopped from the first week in June to the middle of September, which thus gave amply 200 working days in a season. Some of the principal quantities are given in the table.

Principal quantities—Dam

Item.	Cu. ft.
Excavation, all soil, main dam and waste weir foundation	29,80,000
Excavation, rock	38,95,000
Excavation, all soil, waste weir channel ...	1,26,500
Excavation, rock, waste weir channel ...	1,16,600
Building, main dam, cyclopean ...	6,81,000
Masonry, plum concrete hearting ...	41,98,000
Force wall and rubble masonry ...	1,67,69,400
Portland cement concrete in cone wall ...	1,19,200
Waste weir masonry	4,71,100
Top and sundry permanent masonry works ...	26,000
Total Building ...	2,22,64,700

* Tokerwadi dam

A remarkable feature of the lake is the small size of the dam necessary to retain so vast a quantity of water, *viz.*, 14,484 million cubic feet

The site was chosen where the valley was narrow requiring the minimum of masonry. The foundation of the dam is no good and proved trap rock. The formation of the rising ground of the

*The Journal of the Institution of Engineers—J. P. Heath, M.I.E., on Andhra Valley Electric Power Co., Bombay.

valley near the dam has naturally provided for the bye-wash or overflow, the water finding its way again into the Andhra river beds.

The General Dimensions of the Dam.

Height of dam	190 ft.
H. W. L.	2,195 ft.
Draw-off W. L.	2,120 ft.
Sill W. L.	2,117 ft.
Bed of river R. L.	2,005 ft.
Width of dam at base	148 ft.
Width of dam at top	15 ft.
Batter of upstream face	8·4 ft.
Length of dam at top	1,580 ft.
Quantity of masonry	70,300 brass.

The up and down stream faces of the dam are built in coursed rubble masonry, the interior or hearting is built in with trap rock rubble masonry. A water-proof diaphragm is inserted in the dam at the junction of the coursed and rubble masonry up to a certain level to render water-tight the lower portion of the dam, which is under the greatest hydraulic stresses. The masonry of the upper portion of the dam is deemed sufficient as regards water-tightness. The resultant and the dead weight of the dam fall within the sectional middle-third and are maintained.

There is a small auxiliary dam in one place only where it has been necessary to build up a depression in the surrounding hills to maintain the high flood water level of the lake.

Diversion Dam and Intake, Pykara

The main point in the design of the diversion dam is the choice of site. The site must preferably be rocky and admit of the construction of a cheap diverting dam. Probably a narrow part of the river with comparatively calm flow and rocky beds would be found satisfactory. But by far the most important consideration is that the intake must not be affected by the flood conditions of the river. This is accomplished if the course of the river and the nature of flow ensure this condition. A bend in the river course and a sudden drop after the dam may perhaps be most satisfactory. Also the abnormal flood conditions must not be affecting the intake. The intake should be designed such that no logs or other foreign things could enter it. Also it must more or less hold any sediment or suspended matter and allow only water to the aqueduct.

Fig. 31.

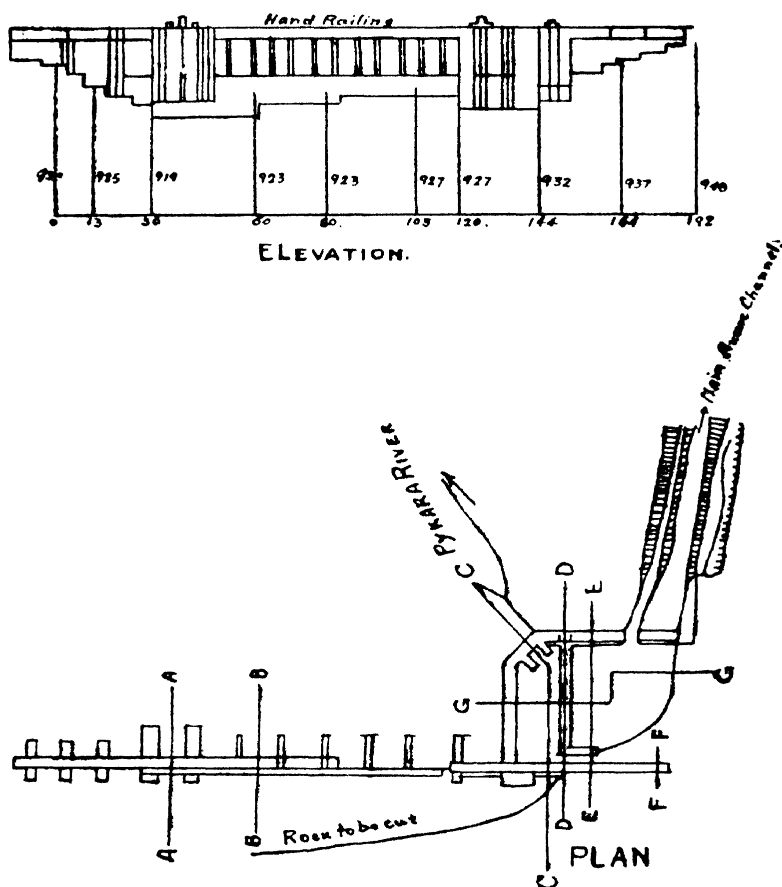

Hand Railing
ELEVATION.
PLAN
PYKARA RIVER
Rock to be cut
Main Channel

Fig. 32.

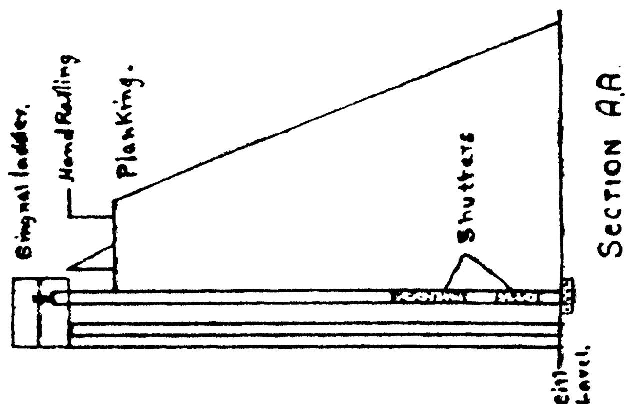
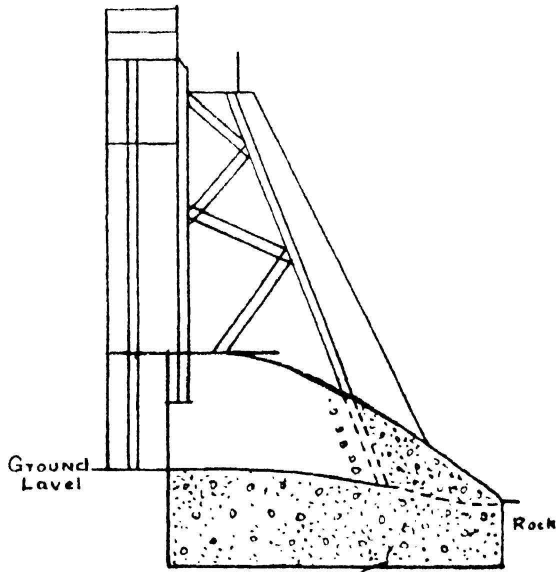
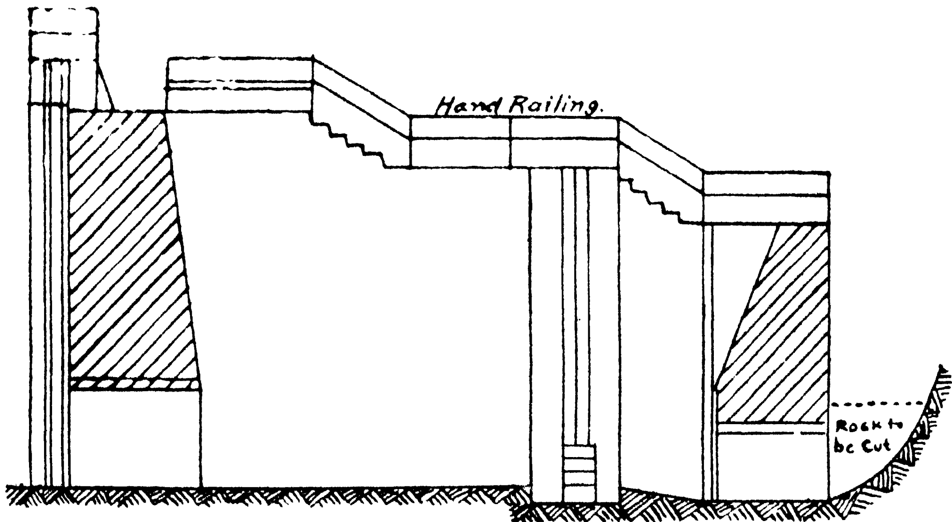

Signal ladder.
Hand Railing
Planking.
Shutters
SECTION A.A.
Sill Level.

Fig. 33.



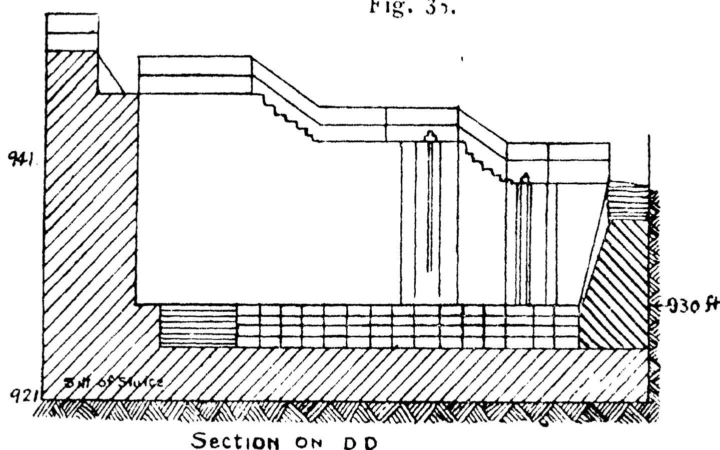
Section on B.B

Fig. 34.



Section C.C.

Fig. 35.



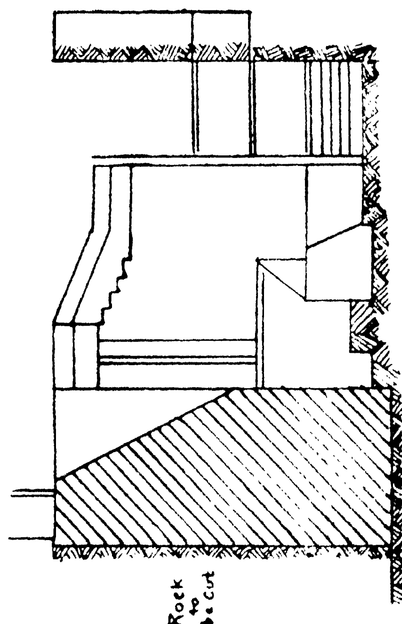


Fig. 36.

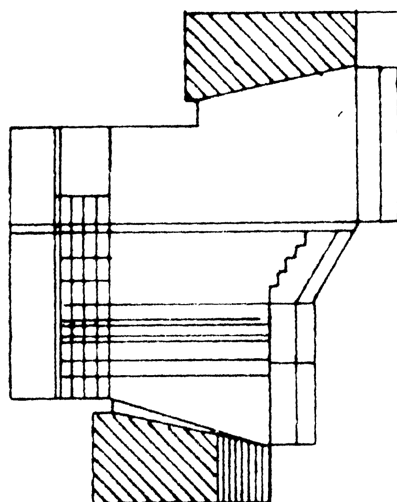


Fig. 37.

The drawing of the " Diversion Weir " gives the constructional details of the diverting dam, intake and main flume of the Main Pykara Scheme with sectional views giving details about the dam, intake, sluices, etc., etc. The constructional details are made clear.

It must be observed here that the design is quite in conformity with the general requirements detailed above. The diversion dam is about a mile and a half above the fall and equally away from the forebay. A site further down would have saved the expense of a long flume But then the river beds would be very steep and would necessitate a bigger dam or excavation of the flume. Excavation for the flume is minimum with the present site though it is a little long. Also the site is a good one with bed rock on both the sides and in the bed of the streams. Just beyond the toe of the structure there is a sudden fall and a change in the course of the river which forms a good "get-away" for any flood water and provides a natural protection to the intake. By means of the two main sluice gates the course of the river can be controlled and deposits of gravel and sand prevented. By virtue of the natural position of the intake side of the site, the inlet weir will never be affected during abnormal flood conditions. Also the intake can be kept clean at all times and no sediment or matter in suspense can enter the main aqueduct.

The diversion wall of the Glen Morgan Scheme is formed only by filling up in between bricks and boulders. The inlet is somewhat similar in its functioning to the main inlet. The crest of the weir is about 1½ ft. below the diversion wall and needs a head of about 1 ft. to supply 27 cusecs. The inlet is protected with screens. The canal level is about 4 ft. below the diversion wall. The first section of the power channel is covered in for about a hundred feet, at the far end of which a spill-way and scour-gate are installed again. Here the design is proof against any flood conditions.

Calculation of the stability of Krishnaraja Sagar Dam.

The section of the dam has been divided into 15 parts by the lines indicating also the width of the dam section, and the centres of gravity G_1, G_2, G_3, etc., of the various parts found. The load line due to the weight W_1, W_2, W_3, etc., has been set down, and the funicular polygon drawn at the bottom of the figure, and the line of thrust, when the reservoir is empty, is drawn from it in the usual way. (Fig. 37.)

When the reservoir is full of water, the magnitude P_1, P_2, P_3, etc., of the water pressure on the water side faces of the dam is determined along with the centres of pressure and the load line drawn, the various water pressures acting, of course, perpendicular to the dam face. From the point of intersection of the water pressure line P_1 on the topmost element of the dam section and the vertical gravity line through G_1, a line is drawn parallel to the full line P_1W_1 on the load diagram, giving the line of thrust for this element. Produce this to cut the water pressure line P_2 at the point A. Then the dotted line AB drawn parallel to the dotted line P_2W_1 on the load diagram gives the line of action of the resultant of the water pressures P_1 and P_2 and the weight W_1 of the topmost element. Let the dotted line AB cut the vertical through G_2 in the point B. Then the line of thrust for the second section, which is the resultant of the water pressures P_1 and P_2 and the weight W_1 and W_2 due to the weight of the dam above the line of 14·5′ forming the base of the second element of the dam section from the top, must pass through this point B. From B draw BC parallel to the full line P_2W_2 on the load diagram to get the line of action of the resultant thrust and let it cut the water pressure line at the point C. From C draw CD parallel to the dotted line W_2P_3 on the load diagram to cut the vertical through the point G_3 in the point. Then the full line drawn from the point D parallel to the full line P_3W_3 on the load diagram gives the line of thrust for the third element of the dam section from the top. In this way proceed till you come to the end. The line joining the various points of intersection of the bases of the various section

elements with their corresponding resultants gives the *line of thrust when the dam is full.*

Height from top in feet.	Width of joint in feet.	RESERVOIR EMPTY.		Distance of C. P. from O. M. T. line	RESERVOIR FULL		Tan θ
		Distance of C.P. from I. M. T. line feet.	Intensity of stress in tons per sq. foot.		Vertical intensity of stress in tons per sq. ft.	Intensity of stress as per b o u n d e r's principle.	
10	12.0	2.00	0.703	1.45	0.903	0.922	0.17
20	14.5	1.33	1.650	1.28	1.70	1.85	0.33
30	19.5	0.90	2.430	1.07	2.39	2.88	0.45
40	25.0	0.17	3.310	1.12	2.94	3.76	0.53
50	31.5	0.14	3.800	1.00	3.50	4.64	0.57
60	38.0	0.29	.290	0.90	4.01	5.44	0.59
70	45.0	0.30	4.700	0.83	4.65	6.32	0.60
80	52.0	0.31	5.350	0.97	5.25	7.20	0.61
90	59.5	0.35	5.600	0.51	5.74	7.95	0.62
100	59.5	0.07	6.420	2.77	5.80	8.00	0.62
110	78.0	0.30	6.790	4.10	5.95	8.15	0.61
120	88.0	0.47	7.250	5.17	6.06	8.25	0.60
130	99.0	0.40	7.650	8.10	6.03	8.11	0.59
140	111.0	0.20	8.120	11.10	5.93	7.95	0.58
150	124.5	0.30	8.390	14.60	5.79	7.67	0.54

The Metur Dam and the rest will be described while examining different schemes.

CHAPTER X

CONDUITS

Definition :—The conduit is the artificial waterway to carry water from the intake. After the water has passed the intake at dam, it enters the conduit and is conducted from there to the turbines at the power house.

Classification :—The following are the general types of the conduit used in the hydro-electric practice :—

Open conduits :—

Canals—lined or unlined.

Flumes—wood, concrete, or steel.

Closed conduits :—

(*a*) For low pressures :—

Tunnels.

Pipes—wood, concrete or steel.

(*b*) For high pressure :—

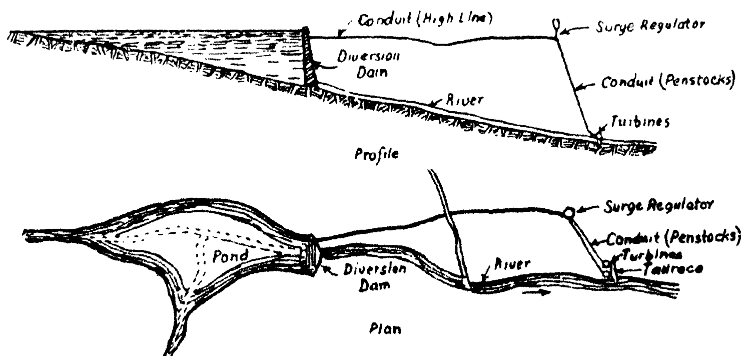
Pipes—steel.

The conduit system may be divided into parts, namely :—

(*i*) the high-line conduit.

(*ii*) the penstock.

Fig. 1.

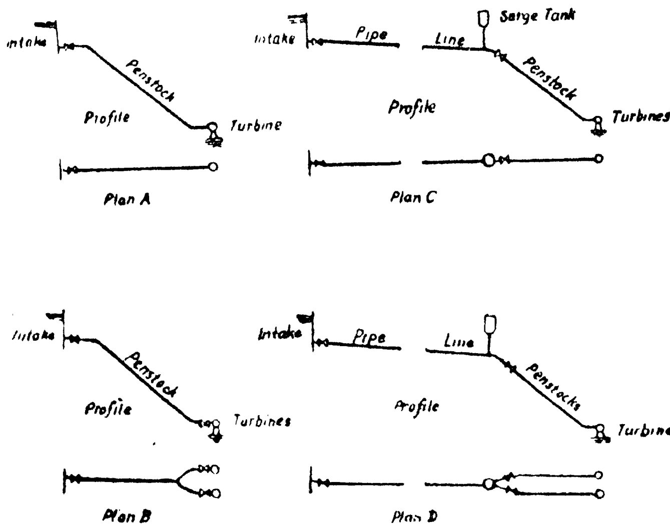


The high-line conduit is that portion which, in general, follows a grade at or as close as possible to the elevation of the low-water hydraulic gradient as shown in Fig. 1. The penstock connects the lower end of the high-line conduit with the turbine as shown. The high-line conduit may consist of a canal, flume, tunnel, one or more pipes, or a counter motion of these.

For the penstock one or more pipes are provided in most cases. Sometimes tunnels are also provided.

Pipe line is the name given to the high-line conduit when it is a pipe. Its object is to distinguish it from the penstock which is a pipe in general.

Fig. 2



In very low-head developments, the penstock is eliminated and the high-line conduit is an open flume in the end of which the turbine is set.

Frequently a high-line conduit is marged with or supplemented by an open body of water at a valley crossing, where it is cheaper to build a dam than to provide a conduit over or around the depression. Such an open body of water is in reality a part of the conduit system.

If the high-line conduit is long, it is usually provided at its lower end with a surge tank, terminal pond or other surge-regulating device or provision is to be made for spilling water when the turbines are suddenly shut down.

Limitations :—Various parts of the conduit system being interdependent the conduits are to be considered in connection with the other features of the project. Thus it may pay to change the site of the dam in order to shorten the length of the conduits. On some projects the required length of the conduit is found to be so expensive that it pays to break up the project into several smaller ones having a very much shorter length of the conduit. A flat country and a low load factor favour the elimination of conduits or their reduction to a minimum and the location of the power house is an integral part of the dam.

Choice of conduits :—The economic consideration will generally govern the choice of a conduit. A thorough investigation should be made of the feasibility of using different types of conduits for the project under consideration and a comparative estimate of cost is to be made for different types of conduits.

Canals are generally constructed for a high rate of discharge, *i.e.*, they are constructed to divert volumes exceeding 500 cubic feet per second. It is cheaper in construction for a high rate of discharge.

The flumes are constructed for a moderate rate of discharge, *i.e.*, about 500 sec.-feet. They are generally built where the contour of the country is very irregular or the soil very hard and difficult to excavate. In very loose soils canals cannot be recommended and flumes can be used in such cases. They have the advantage over canals as high velocities can be used, thereby reducing at the same time the area of its cross-section as the frictional coefficient is less and the velocity of flow is not limited by the physical characteristic of the material.

When the country is so steep and rugged that a conduit could not follow the hydraulic gradient, canals are, of course, ruled out of consideration and the choice lies between pipes and tunnels. Canals or open flumes must be nearly level from end to end, the entrance end being higher than the discharge one by an amount equal to the head necessary to produce the required velocity of flow. Closed conduits may be used to follow any contour within limit. The selection of the pipe may be varied provided it be not raised above the hydraulic gradient.

Where the proposed route of the waterway encounters mountain ridges, it is often advantageous to go through these by means of *tunnels* rather than to excavate deep cuts or go around. The tunnels are necessary in such cases when the altitude to which pipes would have to be carried in order to pass the intervening elevations would rise far above the hydraulic gradient and be too great for them to act as syphons. Compared with well-protected steel or reinforced-concrete pipe, a tunnel offers no advantage and moreover it is expensive. Hence, the tunnel should be driven where it is absolutely necessary and unavoidable.

Compared to open canals and especially if these are built on hill sides where they are exposed to dangers from boulders striking them and undermining, seepage, etc., tunnels are, of course, safer and their upkeep is usually low.

The diversion in pipes is more economical when the volume to be diverted is less than 500 cubic feet per second, or when the construction cost of a canal or flume is abnormally high. In high

speed developments the final passage of water to the turbines is always through the pipes.

In cold countries both canals and flumes, if of great length, are objectionable due to the formation of ice ; pipes seldom give troubles from freezing. Steel pipes, if exposed, are reduced in cross-sectional area due to the formation of a coating on the steel inside it. Freezing in concrete and wood-stave pipe seldom occurs as concrete and wood are better insulators than steel.

Location of conduits :—There are two ways by which the conduits can be located, *viz.* :—(*i*) the field method, (*ii*) the paper method.

The "*field method*" of location of the conduits is similar to the process adopted for the location of a rail road. For this purpose a field party consisting of a locating engineer, a transit-man, a levelman, a draftsman, chainmen, stake-men, etc., is needed. The transit party follows the level party establishing the alignment and stationing the line. The locating engineer may locate several alternate lines if they are feasible.

In the "*paper method*" of location of a conduit is to locate different routes on a topographic map which gives the requisite amount of detail and the necessary field informations as to the nature of the materials to be encountered. For this purpose a topographic map for the entire area through which the conduit is to be driven is studied first for the necessary field information. It is possible to make a paper location of the proposed conduit which is consistent with all the controlling factors of the project. Several alternative conduit lines can thus be laid down on the topographic map, and plotting their profiles and estimating the cost, the most economical route can be dertermined.

The field method of location of a conduit is open to serious objection as all the factors affecting the choice of the conduit location cannot be known to the locating engineer in advance of the general office studies of the project. At first the most economical route is to be determined from the topographic map and then the field party should lay out and stake the line so determined, and obtain a profile and cross-section to substantiate the paper location. After the final location has been made in the field, any changes which then appear to be advisable, can be made.

Velocities and friction head in conduits:—The following points are to be considered in determining the velocity of flow through the conduit :—

(*a*) The allowed friction loss at the full-load, and hence the velocities in the conduit system increases as the head increases. Frictional losses at the full-load vary in general between 5 to 10 per cent. of the gross head, the average being 6 %. Hence, for cases where full load is required for only a very short period each day, a larger loss can economically be allowed than if full load were of long duration.

(*b*) No general statement can be made for velocities in canals, as too many conditions influence the choice. Full-load velocities in high-line conduits usually decrease as the length increases. Excessive frictional loss and the fluctuation of the water surface at the surge regulator must be avoided. The full-load velocities in high-line conduits vary from 5 to 15 feet per second.

(*c*) Full-load velocities in penstocks for low and moderate head plants are effected not only by the consideration of economy but also by the factors of water-hammer and turbine speed regulation. The penstock velocities are never less than 6 ft per second, but it should not be more than 20 ft. second. The average value of it for moderate-head plants is about 10 ft. per second.

(*d*) Very high-head plants are to be provided with automatic by-pass to prevent sudden stoppage of flow at times of flow when the turbine gate is closed. Hence, there is no definite limit to velocity other than that dictated by economy of design.

(*e*) Full-load velocities at the intake gates vary between 2·5 and 8·0 ft. per second with an average of about 5 ft. per second with economy. For successful operations the velocity through the gates for a given maximum discharge at full load may vary between wide limits depending on the head of the development. High-head developments permit of high gate velocities as the percentage of head loss is relatively small. Common velocities are 2 to 4 ft. per second in low-head plants, 4 to 7 ft. per second in medium-head plants, and 7 to 10 ft. per second in very high-head plants.

The local conditions vary so widely that it is impossible to give close limits of the allowed velocity and the head lost in friction. Each problem is to be solved and the judgment of the designer according to the economic theory of design.

Economics of conduits :—High velocity with the resultant small area and small size of the conduit may account for the cheapness at the beginning, but it results in high frictional loss and decreased head and, consequently, the less output. Since initial expenses can be minimised at the sacrifice of the output and the maximum output can be obtained increasing the initial expenses, there is one size of the conduit in every case which gives theoretically the best result. The theoretically best result can be obtained from the principle of economic design.

The following practical points are to be considered in determining the velocity of flow through the conduit :—

(1) In canals of earth the velocity must not be high enough to cause scour or low enough to allow the growth of plant or deposits of silt.

(2) In penstocks the velocity is influenced to a large extent by considerations of turbine regulation. A high velocity, while it may have been proved more economical, may require such a slow-moving governor to prevent excessive water hammer as to result in unsatisfactory speed regulation.

The usual velocities in the conduits are given in the previous section.

In calculating the economic conduits from the principle of economic design, the features to be considered for the conduit are to include the cost of all appurtenances that change with the size of the conduit along with the annual cost. The annual cost of high-line conduit must include the cost of grading, sills, cradles, trestles, terminal regulator and other important appurtenances. The size of the surge tank changes with the velocity of the conduit and, hence, it will have influence on the size of the conduit. The internal pressures in closed conduits and the depth of water in the open conduits have influence on the velocity of flow through the conduit and on the size of regulator and, hence, they should be considered in all problems of economic design.

The cost of dam creating a pond varies inappreciably with the size of the conduit and, hence, the problem of economic design is not difficult for open conduits with ponds for regulators.

Open channels and canals :—Where the distance between the headworks and the power-house cannot be bridged by a pipe line, an open channel or flume is used. On comparatively low heads this may be a canal of large capacity, excavated in earth or rock and, if necessary, lined. Where the required capacity is smaller, an artificial channel of wood, concrete, or metal, is employed either founded on a track in the hill-side or supported on a trestle framework or on pillars. Wood cannot usually be employed in the tropics, even when impregnated or coated. Ordinary masonry is not so satisfactory as concrete, and it should be lined with cement, if used. Circular channels of iron are often employed. Occasionally for very small discharges even rectangular or trapezoidal troughs of thin galvanized iron are used.

An open channel requires very careful setting out before construction, especially if placed on a made track. Catchwater drains are required behind it to prevent a wash-out and also to ensure that ordinarily nothing will fall inside from above. The smallest natural drainages must be properly bridged or carried off in culverts and at these points arrangements should be made for surplussing water from the channel, if it should be necessary for repairs or because an excessive amount has been taken in By raising the height of the sides at all other points, an overflow from these causes, or from the accidental blocking of the channel lower down, can be safely and automatically discharged where the ground is suitable to receive it.

Where two or more streams are tapped separately there is greater security. There are some installations which tap a large number of small streams by independent channels of gradually increasing size as they merge into one another.

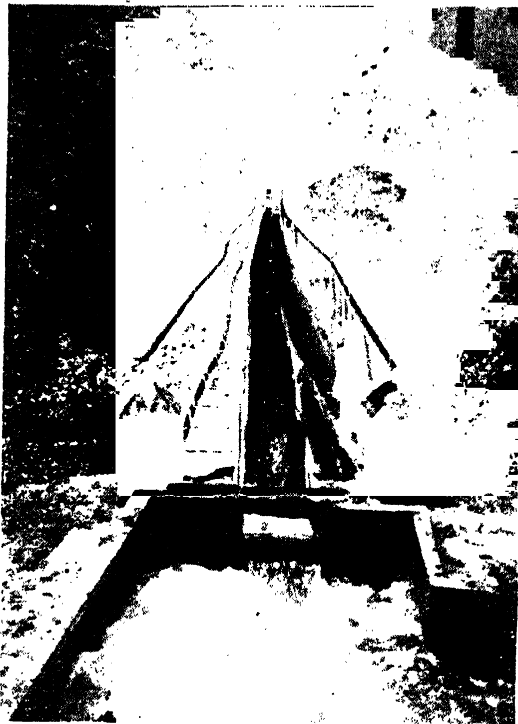
In developing hill streams liable to sudden spates, it is found worth while to make small subsidiary channels to tap any minor streams. Canal or open channel is used (owing to its cheaper construction and greater ease of working) to carry the water on a very small slope up to a point immediately above the power-house side. There the canal discharges into a forebay from which the pipes are taken off. This arrangement is possible only when the ground on one bank at least remains at the necessary altitude. It is generally necessary to provide a silt trap near the headworks, to prevent the channel getting blocked, and another one at its termination, in case foreign matter falls in by the way. A reservoir for regulating storage may be at any site where the ground allows it ; but the nearer it is to the forebay, the better, seeing that trouble may occur on the channel.

In order to secure the safety of the weaker upper sections, to give a stand-by supply of water during the period occupied by the long column of water in accelerating upon an increase of demand, and to provide a relief chamber for the excess, tending to enter the lower sections when the demand is suddenly checked, a *surge tower* or stand-pipe is generally connected to the pipe system at the junction. The reservoir at the top of the surge tower rises to a level slightly above that of the headworks or forebay feeding it, and is provided with an escape ; the necessary capacity of surge reservoir depends on the length and dimensions of the pipe line and on the flow therein. The similarity between the tank of a surge tower and a forebay will be evident. In the rare cases where the forebay is connected to a larger reservoir by a practically level underground pipe, the forebay is actually a surge tower also, over which the open channel may pass, even though these may run quite dry ordinarily. The reason is that during very heavy rains there is always a fair discharge in these small channels, and owing to their small catchment areas it is generally moderately clear; whereas at such times the main stream will be heavily silt-laden and the headworks may be temporarily put out of action.

The expense incurred in crossing minor streams is often considerable. The complete arrangement of such a crossing at Nepal is shown in Plate No. 3. In many cases clear span is impracticable and the bridge must be on piers. In such cases care must be taken that the piers cannot be destroyed by a rock bombardment at flood times.

Canals Section :— Canals in earth generally have a trapezoidal section while canals in rock frequently have very nearly vertical sides, the slope of the sides being determined by the angle at which the rock breaks out most conveniently.

Fig. 4.



Suspension bridge showing the conduit line, Darjeeling.

For a given area of cross-section and given slope, the most economical section, as explained before in hydraulics, is that which gives the maximum discharge. For a trapezoidal section with most economy :—

$$d = \sqrt{\frac{A \sin\theta}{2 - \cos\theta}}$$

$$b = 2\,d \tan\theta/2$$

$$A = \frac{a + b}{2} \times d$$

Hydraulic radius or mean depth = d/2.

Where, A = area of the section.

In rock-cut the sides are vertical when $\theta = \pi/2$ and for as such a rectangular and economic section b = 2 d.

Example :—A canal is to carry 270 cu. ft. per second at a velocity of 3 ft. per second. The side slope is 1·5 horizontal to 1 vertical. Find a suitable section of the canal.

Solution :—

$$A = 270/3 \text{ or } 90 \text{ sq. ft.}$$

$$\tan\theta = \frac{1}{1{\cdot}5} \quad \text{or} \quad \tfrac{2}{3} \quad \therefore \quad \theta = 33{\cdot}7^\circ.$$

$$\sin\theta = 0{\cdot}555$$

$$\cos\theta = 0{\cdot}832$$

$$\therefore \text{ depth } = \sqrt{\frac{90 \times 0{\cdot}555}{2 - 0{\cdot}832}}$$

$$= 6{\cdot}53 \text{ ft. or, say, } 6{\cdot}5 \text{ ft.}$$

width b at the bottom $= 2 \times 6{\cdot}53 \times 0{\cdot}3019$

$= 3{\cdot}95$ ft. or say, 4 ft.

If a = width at the top of the section,

$$\therefore \frac{a + 4}{2} \times 6{\cdot}5 = 90$$

$$\text{or, } a = \frac{180}{6{\cdot}5} - 4 \text{ or } 23{\cdot}7 \text{ feet.}$$

Another practical consideration to be made with the theoretical economic section is that, in a deep rock cut, the theoretical ratio of the bottom width to the depth of water must be reduced to decrease the amount of total excavation. It will frequently be found, also, that the bottom width may more economically be made wide enough for steam shovel and track work, even at the expense of additional excavation.

Similarly, it will be found that deep-cut and hill-side canals in earth will have a most economic section which does not correspond to the economic section calculated from the above equations.

But from the standard point of safety, a shallow canal is better than a deep one, when these are built on the sides of hills as the pressure on the bank increases with the depth of water where the banks may have been weakened by erosion and afterwards may break.

In case of canals lined with rocks, there is only one size for case which will make for the greatest economy of the design. The design is to investigate it from the economic theory of design.

Side-slopes of canals :—The slopes should be such that they will withstand erosion of the water. For canals excavated in earth a slope must be made at an angle which is slightly greater than the angle of repose of the material through which the cut is made.

The following values of side-slopes are the actual practice to withstand erosion of the water :—

	Horizontal.	Vertical.
Solid rock or cement	$\frac{1}{4}$-$\frac{1}{2}$	1
Hardpen and very firm soil ...	$\frac{3}{4}$-1	1
Ordinary firm soil	1	1
Ordinary sandy loam	1·5	1
Loose sandy soil	2	1

The super-banks of the canals should be sloped at two horizontal in one vertical and paved.

Velocity of flow through canals :—The velocity of flow through a canal must have a permissible value so that there may not be any tendency to erosion of the sides and bed. Again it should not fall low enough to permit the deposit of silt or debris. If there be a silt basin, at the conduit at the place from where it is coming out of the reservoir, the condition for the minimum velocity in the conduit is to be neglected. Moreover, the conduit is coming out of the lower end of the reservoir and most of the silt carried by the stream are deposited in the reservoir. A mean velocity of from 2·5 to 3·5 ft. per second will generally be sufficient to prevent the deposit of silt in unlined canals.

If the soil contains a considerable percentage of clay, original velocities as high as 5 feet may be used.

Another point is to be noted that the growth of aquatic plants and moss seriously affects the capacity of the canals. For this reason there must be a mean velocity of flow greater than 2·5 ft. per second which will prevent the growth of aquatic plants and moss and to prevent the deposit of silt.

The maximum permissible velocity in a canal is such a velocity which will not erode the bottom and the sides of the canal. When a canal is lined with concrete, no definite maximum limiting value can be set, provided that the water does not carry sand, gravel, or stones.

For very high velocities over a concrete lining there is a tendency for the rapidly moving water to pick up the blocks and move them out of position, because the pressure under the blocks sometimes exceeds that of the rapidly moving water just above. For this reason when high velocities are in such a channel the concrete blocks are often made several feet in thickness. Also the foundation upon which the concrete blocks are laid should consist of sand, gravel, or soil instead of rock.

To recapitulate.—

(1) It must not be so low as to deposit silt or debris.

(2) It must not be so low as to allow the growth of aquatic plants.

(3) It must not be so high as to erode the sides and bed of the canal.

(4) A mean velocity of not less than 2·5 ft. per sec. will generally prevent a growth that would seriously decrease the carrying capacity of the canal.

Etcheverry gives the following maximum safe mean velocities against erosion for different materials :—

	Velocity in ft./sec.
Very fine sandy soil or loose soil ...	0·5
Pure sand	1·00
Light sandy soil, 15% clay	1·20
Light sandy loam, 40% clay	1·8-2·0
Coarse sand	1·5-2·0
Loose gravelly soil	2·5
Ordinary loam	2·5
Ordinary firm soil or loam, 65% clay ...	3·00
Stiff clay loam	4·00
Firm gravelly clay soil	5·00-7·00
Stiff clay	6·00
Soft slate	6·5
Stratified rock	8·00
Small boulders	8·00-15·00
Hard rock	13·33
Concrete	15·00-20·0

The bottom velocities are approximately 75 per cent. of the mean velocities.

Seepage Losses :—The United States Reclamation services have investigated the following formula for the seepage losses in canals :—

Loss of seepage in cubic feet per second is—

$$\frac{J\,p\,l}{86{,}400}$$

where, p = wetted perimeter in feet.

l = length of canal in feet.

J = a constant, depending upon the character of the material of the canal bed.

The following table gives the value of J :—

Material of the canal bed.	Value of J.
Cement gravel and hard pan with sandy loam	0·34
Clay and clay loam	0·41
Sandy loam	0·66
Volcanic ashes	0·68
Volcanic ashes with some sand	0·98
Sand with volcanic ashes	1·20
Sandy soil with some rock	1·68
Sandy and gravelly soil	2·20

In most power canals the losses due to seepage are rather insignificant. But in canals in arid regions, seepage losses are a matter of great concern.

Other things being equal, a canal in cut will have much less leakage than one having a great deal of enbankment. It is frequently possible to put the canal in cut below normal water surface, and provides embankments to retain usual high water and surges. Leakage is more for a cut on hill-side and especially for canals constructed through previous material.

In order to prevent the leakage, the canals constructed through pervious material must be lined with concrete. The concrete makes one of the most satisfactory linings and is usually laid in thickness of from 4 to 6 inches depending upon the depth of water to be maintained in the canal and the material on which the concrete is to be placed.

Lining of canals :—It has been said before that the object of concrete lining is to prevent the canal from the losses due to seepage, but it also prevents the leakage which would otherwise take place.

The excavation of the canal cross-section for a lined canal will, in most cases, be much less than the excavation of an unlined canal for the same project.

A lined canal has the following advantages :—

(*i*) A lined section will give a much lower value of Kutter's n than an unlined section and hence permits the use of higher velocities.

(*ii*) For a lined section, the velocity of flow will not be limited to the maximum for which the given material is safe against erosion, but will permit the use of velocity giving an economical loss of head.

(*iii*) The higher velocity permits the use of a smaller section.

(*iv*) The maintenance of a lined conduit is much less than that of one unlined.

(*v*) On hill sides a smaller conduit section is very often more practicable to construct, where large conduit may be unsafe or commercially impossible or only possible at excessive cost.

(*vi*) With lined conduit the water reaches the forebay in less time than in the case of one not lined, also the water is more easily disposed of in case of a break in the conduit, and when shutting down for repairs.

(*vii*) The lining of a conduit permits the use of a section with highly desirable hydraulic properties, and different from an earth section, maintains the sides and banks of the conduit so as to retain those qualities.

(*viii*) The lining of a conduit stops erosion of the banks by wave action and delays the growth of weeds, brush, and moss, all of which interfere with the flow of water and increase deterioration.

(*ix*) With an increase in the depth of water in the conduit, the velocity and the carrying capacity are increased. Without lining, an increase in the depth would usually not be permissible.

The following **canal linings** *are generally used* :—

(*a*) Concrete slab lining.
(*b*) Reinforced concrete lining.
(*c*) Timber lining.
(*d*) Clay lining.

(*a*) In concrete slab lining, the concrete is generally mixed in the ratio of 1 : 2 : 4. The concrete blocks are made from 5 to 12 feet square with 4 to 6 inch thick. The object of dividing the lining into blocks is to provide for the possibility of settlement when the blocks would move as units and produce cracks. The division into blocks will also provide for expansion and contraction due to temperature changes. Sometimes these blocks are poured with three-ply tar paper between them, and an expansion joint formed by a $\frac{1}{4}$ to $\frac{1}{2}$ in. wide strip of asphaltic felt or bituminous expansion-joint material is located at every sixth block. These blocks will allow no water to get through their joints when expanded.

Another effective manner of securing water-tight joints is to place a dry yellow pine wood strip $\frac{7}{8} \times 1\frac{1}{2}$ and an expansion joint between adjacent blocks of concrete lining. The dry wood when put in place gradually takes up water swells and becomes very tight in the concrete. Occasionally, where absolute water-tightness and absolute permanency are required, a reinforced concrete seat is provided at the block-joints.

In canals whose side-slopes are flat enough to permit the placing of the concrete without forms, the cost of placing is greatly reduced Unless the canal embankments have been allowed to stand for sometimes before the lining is placed, the bank sometimes shrinks away from the lining at the top. These cracks between the lining and the bank should be carefully sought out and churned full of soft clay, as otherwise the lining may crack when the canal is filled with water.

On side hills or wherever the ground slopes towards the canal, surface ditching should be made to prevent the surface water from collecting behind the lining and from upturning it. A complete drainage ditch should be dug to collect the water and bring it into the canal over the top of the lining.

(*b*) Sometimes when the canal has embankments it is advisable to reinforce the lining with a steel wire mesh or light reinforcing rods instead of using concrete slabs for lining the canal.

As steel is depended on to take of temperature and shrinkage stresses, expansion joints need not be used here. This sort of lining is tighter than one in which slabs are used. In order to get a very smooth surface having a very low value for Kutter's n, it is better to use a fairly stiff mixture and temping it into place. The steel ratio generally used for such linings is about 3/10 of 1 per cent. or less. The concrete mixtures used vary from about 1 : 2 : 4 for the greater thicknesses of concrete to mortars without any stone or gravel for the $1\frac{1}{2}$ inch thicknesses. The mortars used for this purpose vary from 1 to $2\frac{1}{2}$ to as lean as 1 to 5. A mortar as lean as 1 to 5 is not recommended but has been used for this purpose.

(*c*) The timber lining consists of 3 inch planking laid longitudinally upon 12 × 12 inch timber sills spaced 8 ft. centre to centre and imbedded in the bed of the material, being secured in place by connection to bearing pipes from 8 to 16 ft. long or by iron rods. This sort of lining is seldom used at present.

(*d*) Clay lining is sometimes used for the bottom of the canal. It should be used with caution and well protected as it can be eroded at the time when the canal is to be emptied or filled with water.

Ice troubles :—When the canal is shallow and long enough, a great deal of troubles is experienced due to the formation of ice in the cold climates. If the velocity of flow be high, ice forms to a considerable extent and which gives much troubles at the racks and frequently causing the plant to be shut down. Narrow and deep canals are subjected to less ice troubles than a shallow and wide ones of the same capacity. Ice cakes should not be permitted to enter into the conduit from the intake. Provision should also be made in the forebay, near the intake to the penstocks or power house by means of a skidway to take care of cakes of ice.

If any open channel, of any kind, is subjected to freezing, its cross-section should be made of sufficient area to supply the required quantity of water when ice-covered. The cross-section should be computed from the following formula where 'm' is the value of Bazin's constant to be taken into calculation.

$$\frac{p_1 \times m_1 + p_2 \times 0{\cdot}06}{p_1 + p_2} = m$$

Where, p_1 = length of wetted perimeter of the canal bed.
p_2 = perimeter of the ice surface.
= width of the stream.
$p_1 + p_2$ = total wetted perimeter.
m_1 = Bazin's coefficient for the canal bed.

In order to be saved from ice troubles, the velocity of flow should be lowered so that a sheet of ice can be formed on the surface.

Location of canals (*vide* p. 238) :—The location should be economically the shortest one determined by the paper location in a topographical map as explained before. Deviations from a tangent alignment will generally prove justifiable to avoid side-hill cuts, buildings, road crossings, etc., so that the curve may be limited to 3° per 100 ft. length of it.

The slope in curved canals is greater than in straight. Humphrey and Abbott's formula is generally used to give the excess slope, which is as follows :—

$$hc = \frac{6\,d\,v^2}{1683}$$

Where, v = mean velocity.
d = the total angle of the curve expressed in radius.
hc = excess slope.

As for example in a canal, velocity of flow = 5 ft./sec.

Length of the curve = 900 ft.
Curvature = 3° per 100 ft. length.

$$hc = \frac{6 \times 5^2 \times 9 \times 3 \times \cdot 01745}{1683}$$

$$= \frac{70 \cdot 67}{1683} \text{ or } 0 \cdot 042$$

The general slope of the curve = 0·051 per 100 ft. length.
∴ the total slope of the curve channel = 0·051 × 9 + 0·042
= 0·5

i.e., the curved portion of the channel should have a vertical depression of 0·5 ft. in 900 ft. length.

Construction and Excavation :—The construction of a canal depends upon the location and design of the canal pertaining to the excavation of the prism, the lining of the bed and slopes and the rivetting of the super-banks. The excavation cost depends upon the character of the material, the quantities to be removed, and the deposition which may be made of the spoils ; all these together with the depth and width of the cut will influence the methods which secure the most economical excavation. Hard rock can be excavated by the use of explosives, drilling, blasting, loading and disposing.

Rock drilling is generally done by machine drills operated by steam or compressed air.

The breaking of the rock for the purpose of loading for removal by means of explosives requires 1 to 1·5 pounds of 60 per cent. dynamite according to its hardness per cubic yard of the output.

Earth can more cheaply be removed by dredging than by dry excavation. Dry earth excavation may be done by hand tools, horse and power scrapers or by steam shovels, the operation consisting of loosening, loading and disposing.

Appurtenant structure to diversion canals :—These are required to safeguard the super-banks against erosion from surface run-off, interception of unavailable lateral stream sources, the devices controlling the flow in the canal and means of overhead crossings. The following form the important part.

(*i*) *Canal spill-ways* :—With long canals when side streams are allowed to discharge into the canal, abnormal surges may take place which must be taken care of without a material rise in the water surface of the canal by means of spill-ways along the canal. Such spill-ways are usually small overflow masonry dams.

(*ii*) *Pond or forebay* is likewise arranged at the lower end of the canal, being simply a gradual enlargement in which the velocity of flow is reduced before the water enters the power-house. This is able to act as regulator to supply the peak load demanded, allowing the canal above them to supply the average rate of flow.

(*iii*) Bridges are often required across the canal for operating purposes and to accommodate public traffic.

(*iv*) A waste weir should be arranged near the end of the canal, being practically a short open spill-way with one or two overflow sluices ; floatage, ice and surplus flow may be passed over it.

(*v*) When there is a terminal pond or a large forebay, the velocity through them is so low that sand of any size to injure the turbines is deposited. If there be no forebay, the sand in suspension will be carried to the turbines due to the high velocity of flow, when the resulting wear on the sealing rings of the turbines may be a serious matter, increasing clearances and decreasing efficiencies. To obviate this difficulty the use of sand boxes and settling basins has been introduced.

Troubles and Remedies for Canals :—

(1) Troubles—Canals having a low velocity of flow.
Remedy—The canal should be lined with concrete so as to prevent leakage, give a smooth surface and allow a high velocity of water through them without too great loss in the head.

(2) Troubles—Causing of scours in the canals.
Remedy—Lower the velocity of flow.

(3) Troubles—Growth of plants and the deposit of silts in the canals.

Remedy—Increase the velocity of flow in the canals. A mean velocity of not less than 2·5 ft. per second will generally prevent the growth that could seriously decrease the carrying capacity.

(4) Troubles—Formation of ice in the canals.
Remedy—Lower the velocity of flow so that an ice sheet may be formed on the surface. In order to permit the formation of an ice sheet in the canal, the velocity of flow should be less than 1·5 ft. per second. Once an ice sheet has been formed on a power canal, there is seldom any further trouble from frazil or anchor ice. Generally the hydro-electric plants have light loads at nights when the

velocity of flow can safely be lowered for the formation of ice and if they are to supply any high loads at nights it is frequently desirable to shut the plant for a night or two in order to permit the ice sheet to form.

(5) Troubles—Seepage of water in the canals.
Remedy—Line the canal with concrete.

(6) Troubles—Cracks in concrete lining of canal.
Remedy—Make concrete blocks to be used 5 to 12 ft. square which will move as units and not produce cracks. Also use expansion joints of bituminous felt among the blocks.

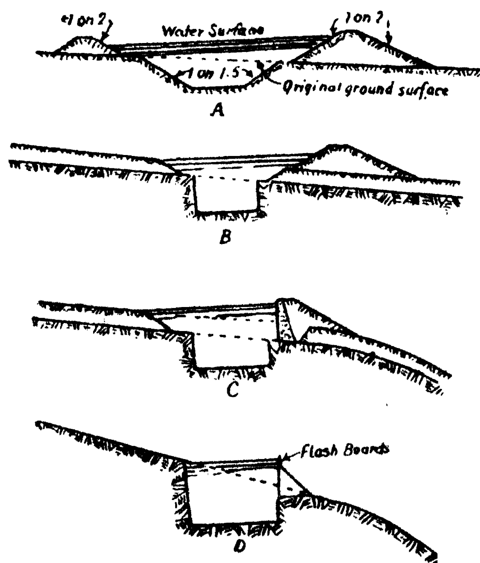
(7) Troubles—Disintegration of concrete linings.
Remedy—Use reinforced concrete linings.

(8) Troubles—The concrete blocks may be picked up and moved out of position in the concrete lining.

An inspection track is always required along the channel, whether at the side or over the top of it for the daily inspection of channel and removing any trouble.

Shape of canal section :—As already described in page 380, canals in earth generally have a trapezoidal section, Fig. 5 (A), whereas the canals in rock have sides which are nearly vertical, 5 (B), the slope to be determined by an angle at which the rock breaks out most conveniently.

Fig. 5.



When sufficient rooms for an embankment is not available, canal walls are to be used, as shown in Fig. 5 (C). When the side is such that it has no back-fill for the canal, a dam is to be constructed, as shown in Fig. 5 (D). It is really a dam as it has no back to fill.

Circular channels of iron are often employed and occasionally for very small discharges even rectangular or trapezoidal troughs of galvanized iron are used.

Flumes

Types :—Open flumes are simply elevated aqueducts made of wood, steel or reinforced concrete supported on or above the ground surface. Their use is principally in connection with canals for the purpose of avoiding long contours which would have to be made if the canal followed along the path of the natural contour. They are also used to carry water across depressions or ravines from the end of a canal on one side of the ravine to the other end of the canal on the opposite side.

Entry-way :—As the cross-section of a flume is less than that of a canal for the same quantity of discharge due to the lesser coefficient of friction and the unlimited velocity of flow for the physical characteristic of the material.

When a canal discharges into a flume, or vice versa, the cross-section of the flume is less than that of the canal ; gradual approaches by curves are to be made so that the water from the canal section is gradually contracted instead of being abruptly changed when the loss of head due to sudden change of velocity will be eliminated.

When water emerges from the flume at a given velocity into a channel having a greater cross-section with an easy approach between the flume end and the connecting canal end, there is a definite pressure head added to the water due to the reduction of velocity as the loss due to sudden change of velocity being eliminated, the loss of kinetic energy is converted into the pressure head of the water. Under this condition water can be made to run uphill and still maintain the velocity required in the canal, provided the rise in elevation is made from the end of the flume, where higher velocity begins to diminish, to the end of the approach in the canal at which point the lower velocity is finally attained

The height to which water can be made to rise under this condition is given by :—

$$h = \frac{v_1^2 - v_2^2}{2g}$$

where, v_1 = velocity of water in feet per second in flume.
v_2 = velocity of water in feet per second in the canal.
h = rise of water in feet.

As for example consider that the velocity of water in the flume is 10 feet per second and that at the canal is 4 feet per second.

$$\therefore\ h = \frac{100 - 16}{64}, \text{ or } 1{\cdot}31 \text{ feet.}$$

i.e., water can be elevated to a height of 1·31 feet.

The simplest and most durable method of making a proper glared or tapered entry-way is to construct it of concrete, the end of the flume being set into the concrete, and making a permanent water-tight joint. This form of connection is suited to metallic, wood or concrete flumes. It provides the proper form of entry and largely removes the difficulty of making an impermeable joint between the bed of the canal and the flume end.

Again if the water enters into the flume from a canal, it becomes necessary to provide a sufficient drop in the upper end of the flume for the increased velocity head. This drop of head is equal to—

$$\frac{v_1^2 - v_2^2}{2g}$$

where, v_1 and v_2 be the velocities of water in the flume and the canal, respectively.

Classification of flumes :—According to the material of which they are built, the flumes may be classified as follows :—

(*a*) Rectangular wooden flumes.
(*b*) Semicircular wood-stave flumes
(*c*) Reinforced concrete flumes.
(*d*) Steel flumes.

The theoretical economical section of any open conduit is semicircular, but for wood and concrete the labour cost for making a semicircular section exceeds the value of the saving in material compared with a rectangular section. Semicircular wood-stave flumes are sometimes installed. Sheet steel flumes have a semicircular section and are therefore subjected to a small loss of head.

Wooden flumes :—The wooden flumes are used only when the low first cost is of prime importance. They have a comparatively short life. Their yearly cost of interest and maintenance is less than the yearly cost of more expensive and permanent structures due to high annual interest charge.

The rectangular wooden flumes should be lined with the timber of good quality free from warps and knots. The thickness of the lagging should not be less than 1½ inches except for very small flumes. The side of the lagging in contact with water must be planed in order to reduce the friction.

Of the various forms of the joints the butt lagging with splines when properly constructed is most desirable. The watertightness at the joints is sometimes secured by battening or by

calking with oakum and pitch; but splines are very effective. Galvanized iron splines are used at present instead of wooden ones especially for the end joints.

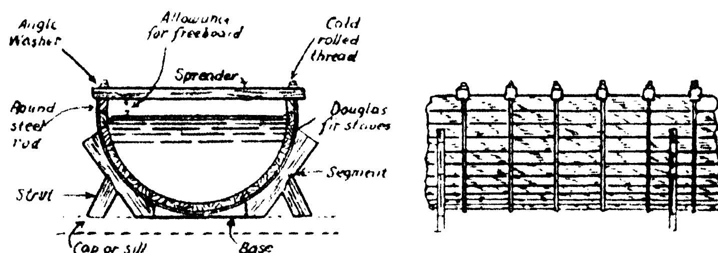
The frame-work of the flumes consists of vertical studs to which the side laggings are nailed, and horizontal sills to which the floor laggings are nailed. The sills are to be extended beyond the studs in order to support the inclined bracers which have the other ends braced against the studs. The studs are to be secured by tying across the top of the box either by a rod or by a timber nailed to the studs.

If the flume is on a bench, cut in a hillside, the sills may rest directly on the ground or on blocking. Broken stone, or coarse gravel under the sills, to facilitate drainage, prolongs their life.

Wood-stave flume:- Semicircular wood-stave flumes are used extensively in many countries for the conveyance of water. The flume-supporting timbers should be made larger than that necessary to support the load safely, owing to the possibility of decay and the consequent weakening of the supports. Rough timbers cut in the vicinity of the work may be used instead of dimension timber, saving time and money. Such timbers are more durable than the sawed ones.

In some cases metallic flumes supported on trestle work are used.

Fig. 6.



Reinforced concrete flume:—Reinforced concrete flumes are the most expensive type, but they are the most permanent and satisfactory engineering construction. In designing such a flume the following points are to be considered :—

(*a*) The safe limiting stresses for this work should be taken as—

Concrete compression	...	500 lbs. per sq. inch.
Concrete shear	...	50 lbs. per sq. inch.
Steel tension	...	18,000 lbs. per sq. inch.

(*b*) Hydrated lime or some equivalent water-proofing should be used in the concrete forming the trough.

(*c*) The stresses acting against bursting or against the sides turning outwards must all be taken by the vertical braces surrounding the trough at each support.

(*d*) There is a transverse flume tending to break the bottom trough. Hence the bottom must be transversely reinforced.

(*e*) The side walls have the upper edges heavily reinforced to act as beams between supports, resisting the horizontal flexure due to side thrust, while at the bottom, where they join to the trough-bottom, they are anchored by short tension rods.

In this way, the sides are supported against bursting outwards at the top and bottom. Hence, each vertical element of the side wall is a simple beam, supported at each end and having the equivalent of the concentrated load at a point one-third of the depth from the bottom.

This condition requires that the vertical reinforcing rods be placed near the outside surface of the walls.

(*f*) The bottom of the side walls are thickened in order to give a greater lever arm to the steel as the side walls are held against both bursting and overturning by cantilever rods. Hence, the upper edges of the side walls need not be reinforced.

(*g*) The distance apart between the supports must not be more than 30 ft. or less than 12 ft. It depends upon the cost of concrete materials, steel, labour and the height of the flume above the ground.

(*h*) The provision for expansion joints must be made at an interval not exceeding 100 ft. apart, otherwise the trough will crack. Each joint must be made at a support so that each adjacent end of the trough should have bearing of at least 8 inches on the support.

(*i*) The support underneath the joints are generally wider than the other ones.

(*j*) The flume will act as a continuous beam and the maximum bending moment between any two consecutive intermediate supports is given by :–

$$M_1 = \frac{WL^2}{12} \text{ ft.-lbs.} = WL^2 \text{ lb.-in.}$$

where, W = weight in lbs. per foot run of the flume when filled with water.

L = distance in feet between the supports.

The average weight of concrete per cubic foot is 145 lbs.

$\therefore W = 145 v + 62{\cdot}5\, bd.$

where, v = volume of the concrete per foot run of the flume, in cu. ft.

b = width of the flume-section in feet.

d = depth of water in flume, in feet.

(*k*) The area of the steel for longitudinal flexure between supports is given by:—

$$A_1 = \frac{8\,WL^2}{7\,f\,(h-2)} \text{ sq. inches} \quad \dots \quad \dots \quad (I)$$

where, f = stress in lbs. per sq. inch in the steel.

h = height of the side wall in inches.

And in this formula A_1 assumes that the reinforcing bars are placed with their centres $1\frac{1}{2}$ inch from the surface of their walls, which are in tension, and 2 inches from the tension surfaces in the thicker portions of the concrete.

(*l*) If the flume acts as a simple beam instead of being a continuous one, *i.e.*, at the two ends or where the spans are adjacent to expansion joints, the spacing can be 90 per cent. of the intermediate spans or the steel must be increased 20 per cent. above the amount calculated from the formula given in (*k*), as the beam will act as a cantilever being supported at the free end, the maximum bending moment being $\frac{WL^2}{10}$ instead of being $\frac{WL^2}{12}$ foot-lbs.

(*m*) The longitudinal reinforcing steel at the bottom must be bent upon the top of the trough section where the flume is continuous across the supports.

(*n*) For the bottom of the trough the *bending moment* per foot length of the flume, of the transverse flexure tending to break the bottom of it, is given by—

$$M_2 = \frac{3W_1 b}{2} \text{ lb.-inch.}$$

where, W_1 = weight per foot length of water and bottom trough.

b = inside width of flume in feet.

Hence, area of the transverse steel per foot length is given by—

$$A_2 = \frac{8 \times 3\,W_1 b}{7 \times f\,(t_2 - 1{\cdot}5)} = \frac{1{\cdot}71\,W_1 b}{f\,(t_2 - 1{\cdot}5)} \text{ sq. inch.}$$

where, t_2 = thickness of the bottom in lbs. per sq. inch.

(*o*) The spacing S of the steel bars—

$$S = \frac{12 \times A_0}{A_2} \text{ inch.}$$

where, A_0 = area of one bar.
and A_2 = total area of steel required.

(*p*) The force acting outwardly at the top side of the wall per foot length—

$$F_1 = 10{\cdot}4\ d^2 \text{ lbs.}$$

The moment of $F_1 = F_1 L^2$
$= 10{\cdot}4\ d^2\ L^2$ lb.-in.

The area of steel required—

$$A_3 = \frac{10{\cdot}4\ d^2\ L^2 \times 8}{7 f (t_3 - 1{\cdot}5)} = \frac{11{\cdot}9\ d^2\ L^2}{f (t_3 - 1{\cdot}5)}$$

where, t_3 = thickness of the side walls.

(*q*) The force F_2 at the bottom side of the wall per foot length—

$$F_2 = 20{\cdot}8\ d^2 \text{ lbs.}$$

This stress is simply tension as it has no lever arm.

∴ area of the anchoring steel at the bottom—

$$A_4 = \frac{20{\cdot}8\ d^2}{f} \text{ sq. ins.}$$

(*r*) *Flexure in the side walls* :—The total pressure on the side walls is $\frac{62{\cdot}5\ d^2}{2}$ per foot length. In practice, it is sufficient to consider that this pressure is uniformly distributed over the sides.

∴ maximum bending moment $= \frac{6{\cdot}5\ d^2 L^2}{16}$ ft. lbs.

∴ area of the steel required is $= 46{\cdot}8\ d^3$ lb. ins.

$$A_5 = \frac{53{\cdot}5\ d^3}{f(t_3 - 1{\cdot}5)}.$$

Cantilever moment for vertical braces at each support :—

The total force acting at the top of the trough
= outward pressure for one span only.
$= 10{\cdot}4\ d^2 L$ lbs.

∴ the cantilever moments $= 12 \times 10{\cdot}4\ d^2\ L \times d$
$= 124{\cdot}8\ d^3 L$ lb. ins.

But in practice the depth of the side wall is divided into two parts each equal to $\frac{d}{2}$.

$\therefore$ the cantilever moment about the middle

$$= \frac{124 \cdot 8}{8} d^3 L \text{ lb. ins.}$$

and the area of the steel required :—

$$A_{61} = \frac{17 \cdot 8 \, d^3 L}{f\,(t-2)} \text{ sq. inch.}$$

and this area extends from top to the bottom of the side walls. Area of the steel required for the bottom section.

$$A_{62} = \frac{142 \cdot 6 \, d^3 L}{f\,(t-2)} \text{ sq. inch.}$$

The above formulæ are used for the type of the flume shown in Fig. 6 where the stresses acting against bursting or against the sides turning outwards are all taken by the vertical braces surrounding the trough at each support. But there is another type where the side walls are held against bursting and overturning by cantilever rods, the bottom of the side walls being thickened in order to give a greater lever arm to the steel. This sort of design avoids the steel reinforcement in the upper edges of the side walls.

For the latter type the area of the cantilever steel will be different and which will be—

$$A_{61} = \frac{17 \cdot 8 \, d^3 L}{f\,(t-1 \cdot 5)}$$ sq. inch per foot length of the trough for the top section.

$$A_{62} = \frac{142 \cdot 8 \, d^3}{f\,(t-1 \cdot 5)}$$ sq. inch per foot length of the trough for the bottom section.

Precautions to be taken in design :—In designing reinforced concrete flume, the ratio of the thickness of the concrete and the area of the steel may be varied according to the local condition. Where steel is costly and concrete is cheap, thick walls will be economical, and if reverse be the case, thin walls with the greater quantities of steel will be economical.

The area of the steel must, on no account, be greater than 0·4 per cent. of the area of the concrete perpendicular to the axis of the reinforcing steel bars, otherwise too great a compression stress may be set up in the concrete.

Example :—In a conduit, $d = 4{\cdot}5$ ft.
$b = 8$ ft.
$h = 5$ ft. $= 60$ inches.
$L = 20$ ft.

Take 0·5 to be the average width of the concrete.

$V = 0{\cdot}5 \times (8 + 10)$ or 9 cu. ft.

$\therefore \quad W = 145 \times 9 + 62{\cdot}5 \times 8 \times 4{\cdot}5.$

$= 3{,}555$ lbs. per foot length.

(1) For longitudinal flexure :—

$$A_1 = \frac{8 \times 3{,}555 \times (20)^2}{7(60-2) \times 18{,}000} = 1{\cdot}56 \text{ sq. inch.}$$

Taking $\frac{1}{2}''$ sq. bar, area of each bar section is 0·25 sq. inch.

$$\therefore \text{ number of bars required} = \frac{1{\cdot}56}{0{\cdot}25} \text{ or 6 approx.}$$

The location of the bars should be three on each side. In order to make the girder a continuous one the rods are bent over the support up to the top and additional rods being given between the ends at the top to give the proper area of the material. But in order to prevent the trough from cracking only two rods, one on each side, are to be bent above the support providing the additional rods between them at the top.

(2) For the transverse stress at the bottom

$$W_1 = 62{\cdot}5 \times 8 \times 4{\cdot}5 + 8 \times 0{\cdot}5 \times 145.$$
$$= 2{,}975 \text{ lbs.}$$

$$\therefore \quad A_2 = \frac{1{\cdot}71 \times 2{,}975 \times 8}{18{,}000\ (6 - 1{\cdot}5)} = 0{\cdot}502 \text{ sq. inch.}$$

where, $6'' =$ thickness of the concrete.

Using $\frac{1}{2}''$ sq. rods the spacing will be :—

$$\frac{12 \times 0{\cdot}25}{0{\cdot}502} \text{ or 6 inches.}$$

(3) For the top edges :—

$$A_3 = \frac{11{\cdot}9 \times (4{\cdot}5)^2 \times (20)^2}{18{,}000\ (6 - 1{\cdot}5)} \text{ or } 1{\cdot}19 \text{ sq. inch.}$$

Using $\frac{3}{4}''$ bars the number of bars required $= \frac{1{\cdot}19}{0{\cdot}5625}$ or 2.

(4) Area of the anchoring steel required :—

$$A_4 = \frac{20{\cdot}8 \times (4{\cdot}5)^2}{18{,}000} \text{ or } 0{\cdot}0233 \text{ sq. inches.}$$

The bottom transverse rods will turn up to the sides and the area of it is greater than A_4 and hence no provision is to be made for anchoring rods.

(5) To prevent the flexure or bulging of sides :—

$$A_5 = \frac{53{\cdot}5 \times (4{\cdot}5)^3}{18{,}000(6 - 1{\cdot}5)} \text{ or } {\cdot}06 \text{ sq. inch per foot length of the flume.}$$

This amount of steel being too small to be conveniently placed in bars, a heavy wire-mesh reinforcement will be the best form of reinforcement and will keep the concrete from cracking.

(6) The vertical wall braces at the support are made tapering towards the top. Take these to be 15 inches thick at the bottom and 5 inches at the top, and the thickness half-way up the braces are 10 inches thick.

$$\therefore \quad A_{6_1} = \frac{17{\cdot}8 \times (4{\cdot}5)^3 \times 20}{18000\ (10 - 2)} \text{ or } 0{\cdot}254 \text{ sq. inch.}$$

Use two $\frac{3}{8}''$ square bars and these have a total area of 0·282 inches. These bars will run from the top up to the bottom.

(7) Area of the cantilever brace at the bottom is given by:—

$$A_{6_2} = \frac{142{\cdot}6 \times (4{\cdot}5)^3 \times 20}{18000\ (15 - 2)} = 1{\cdot}20 \text{ sq. inch.}$$

$\therefore$ Extra area of the steel required at the bottom.

$$= A_{6_2} - A_{6_1}$$
$$= 1{\cdot}20 - 0{\cdot}254$$
$$= 0{\cdot}946.$$

Use four $\frac{1}{2}''$sq. bars for this reinforcement and these bars will extend from the bottom up to the middle.

(8) If the side walls be held against bursting and overturning, take the thickness of the side wall to be 5 inches at the top and 9 inches at the bottom when the mean at the middle being 7 inches.

$$A_{6_1} = \frac{17{\cdot}8 \times (4{\cdot}5)^3}{18000\ (7 - 1{\cdot}5)} \text{ or } 0{\cdot}022 \text{ sq. inches per foot length.}$$

Use heavy wire-mesh for this.

$$(9) \quad A_{62} = \frac{142 \cdot 6\ (4 \cdot 5)^{2}}{18{,}000 \times (9 - 1 \cdot 5)} = 0 \cdot 112 \text{ sq. inch.}$$

The heavy wire-mesh for the reinforcement of the upper portion of the walls will also be sufficient for the lower portion also.

The above data has been calculated without considering the local condition for economy and the ratio of the area of the steel to that of the concrete may be changed according to the local condition.

Steel flumes :—The steel flumes are constructed in various forms, *viz.* :—

(*a*) Semicircular type.
(*b*) Semicircular or half-oval shaped bottoms with vertical sides.

The latter type is most generally used where the vertical sides are designed to carry the load of the flume and its contents, and also to carry the water pressure. The sides are to be tied with steel rods or angles, otherwise they are to be designed to act as cantilevers.

Pipe Lines and Penstocks

It has been explained before the difference between the pipe lines and penstocks and their advantages over other form of flumes also. The pipe lines are constructed of *steel, wood* or *concrete* (reinforced) or *cast iron*.

The cast iron pipes are seldom used for water-power conduits owing to the fact that the diameters required are, usually, great in excess of the practicable diameters. Also the weight of cast iron, its cost of, and the difficulty of handling and joining the lengths together greatly exceed the corresponding characteristics in pipes of other materials.

(*a*) The cast iron pipes are still a first favourite with water work engineers, for moderate heads, and specially for pipes below the ground. For small water-power schemes cast-iron pipes can still be regarded as very useful, because they are easily obtainable, comparatively cheap, and a skilful designer can usually avoid expensive materials.

It has got another advantage that it can resist well the corrosive action. With elaborate methods of casting and testing, by expert foundry-man, cast-iron pipes cannot be made to

preclude the risk of bursting as cast iron is an unreliable material. Apart from the castings being sound, the main point is for the walls to be of correct thinkness, for flanges to be square with the body of the pipe, and of ample thickness with well-rounded fillets. Large flanges may require to be ribbed. The computation for the thickness of the cast-iron pipes is given by :—

$$t = \frac{P + p}{3300} + \frac{1}{4}$$

where, t = thickness of the pipe in inches.

P = maximum steady pressure.

p = allowance for shock.

(b) In water-power practice p may be taken as 3P/4. Flange thicknesses, socket-depths, and sizes in the case of spigot and socket, pipes, bolt holes diameter, etc., are all now fixed by accepted standards.

The cast-iron pipes have really but a limited use in water-power works, being confined to small lines working at moderate pressures and to power house lay-out work when it is suitable for that purpose.

Energy in flowing water :—A mass of water at a given elevation has a certain potential energy with respect to some lower level. In course of its progress from the upper level to the lower one it is divided into the following forms :—

(1) Energy required to produce the initial velocity of flow in the conduit or pipe.

(2) Energy required to move the water into the mouth of the orifice through which it passes in going from the reservoir to the conduit or pipe that carries it to the lower level.

(3) Energy lost in friction due to the passage of water through the conduit or pipe.

(4) Energy lost due to bends, contraction and enlargements of pipe sections.

(5) Energy utilised by the water-wheels such as turbines for useful purposes.

(6) Energy remaining in the water coming cut of the turbines in the form of kinetic energy.

Head :—The total head or gross-head is the difference between the elevation of water in the forebay and the tail race.

Effective or net head = total head—head lost in friction.

The elaborate description of the head lost in friction has been given in Chapter VI dealing with hydraulics.

*__The Mississippi formula__:—In investigating the waterway for the important bridges, the Mississippi formula for velocity is usually adopted in Railways in preference to Kutter's formula which gives results too low, and Dicken's formula which gives results too high. All the road bridges in the vicinity of the alignment are examined and their adequacy or otherwise is taken into consideration in deciding the requisite waterway. The formula is as follows :—

$$V = \left[\sqrt{\cdot 0081\,K + (225\,R\sqrt{s})^{\frac{1}{2}}} - \cdot 09\sqrt{K}\right]^2$$

Where, V = mean velocity in feet per second.

A = average area of three cross-sections of the stream in square feet.

P = average wetted perimeter in feet.

W = average width of water surface in feet.

R = hydraulic mean radius = A/(P + W).

D = hydraulic mean depth = A/P.

S = slope of channel.

$$K = \frac{1\cdot 69}{\sqrt{D} + 1\cdot 5}$$

Note the difference between R and D.

Example 1 :—The Sai river is bridged about $1\frac{1}{2}$ miles above the railway crossing at Rajghat by the Bais bridge of 5 arches of 29 feet span each on the Rai-Bareli-Unao metalled road giving a total waterway of 145 lineal feet, and also about 75 miles further down on the Chord Line, Oudh and Rohilkhand Railway, by 4 spans of 100 feet girders. It is proposed to bridge the Sai at the Railway crossing with 4 spans of 60 feet girders = 240 lineal feet of waterway. There is an old Nawabi bridge about a quarter of a mile above the crossing with a waterway of only 125 lineal feet, in short irregular spans, but it is said that in years of exceptional flood, the water tops the parapet of this bridge. Determine the velocity and the afflux before and after.

Solution : —

A = 11,093 sq. ft. ; P = 1,149·7 ft. ; W = 1,136 ft.

R = 4·853 ; D = 9·65 ;

S = 0·82 feet per mile = 0·000155.

$$K = \frac{1\cdot 69}{\sqrt{D} + 1\cdot 5} = 0\cdot 33$$

* Allahabad, Rai-Bareli, Cawnpore Railway Report and Estimate, E. I. Railway, 1907-1908.

Using the Mississippi formula for velocity—

$$V = \left[\sqrt{\cdot 0088\,K + (225\,R\sqrt{s})^{\frac{1}{2}}} - 0\cdot 09\sqrt{K}\right]^2$$

$\therefore$ V = 3·5 feet per second.

Q = The discharge = A V = 38,826 cu. ft.

Now, a = the new area of cross-section given by the proposed Railway bridge = 5,505 sq. ft.

The new velocity $V = \frac{1\cdot 1\,A \times V}{a} = 7\cdot 76$ ft per second.

The afflux (*i.e.*, the rise of tail race water)

$$= \left(\frac{V^2}{58\cdot 6} + 0\cdot 05\right)\left\{\left(\frac{A}{a}\right)^2 - 1\right\} = 3\cdot 05 \text{ feet.}$$

As the present known H. F. L. = 343·88, the new H. F. L. will be 346·93. Formation level has been fixed at 356·00 ; hence the underside of the girders will be 3·57 feet above the water during a full flood. The abutments and piers will be on wells 16 feet diameter sunk to an average depth of 50 feet below low water level.

Example 2.—The Lone Nuddee is crossed by the Rai Bareli-Unao Road about half a mile downstream of the Railway crossing with a masonry bridge of 3 spans of 33 feet gaining a total waterway of 99 lineal feet. From enquiries made locally the flood water never topped the bridge. It has a well-defined channel, but in the cold weather the bed is dry, it drains an area of 177 square miles. Determine the velocity and the quantity of discharge before and after.

Solution.—

Adopting the Mississippi Formula for velocity—

$$V - \left[\sqrt{\cdot 0081\,K + (225\,R\sqrt{s})^{\frac{1}{2}}} - \cdot 09\sqrt{K}\right]^2$$

Where, V = mean velocity in feet per second.

A = Average area of three cross-sections of stream = 2,891 s. ft.

D = average wetted perimeter = 624 feet.

$$R = \text{hydraulic mean radius} = \frac{A}{P+W} = \frac{2891}{1244} = 2\cdot 324$$

$$D = \text{hydraulic mean depth} = \frac{A}{P} = \frac{2891}{624} = 4\cdot 623$$

S = slope of channel = 1·49 feet per mile = ·00028

$$K = \frac{1 \cdot 69}{\sqrt{D + 1 \cdot 5}} = \frac{1 \cdot 69}{\sqrt{6 \cdot 133}} = \cdot 682.$$

V = 2·72 feet per second.

Q = the discharge AV = 7863·5 cu. ft. per second.

It is proposed to bridge the Lone at the Railway crossing with 3 spans of 4 feet girders = 120 lineal feet of waterway.

A = the new area of cross-section given by this bridge = 2486·71 s. ft.

$$\text{The new velocity } V = \frac{1 \cdot 1\ A\ V}{a} = 3 \cdot 48 \text{ feet per second.}$$

$$\text{The afflux} = \left(\frac{V^2}{58 \cdot 6} + 0 \cdot 05\right)\left[\left(\frac{A}{a}\right)^2 - 1\right] = 0 \cdot 21 \text{ feet.}$$

As the present known H. F. L. is 368·8, the new H. F. L. will be 369·01. Formation level has been fixed at 79·00 ; hence, the underside of the girders will be 6·52 feet above the water during a flood. The pier and abutments are founded on clay and kunkar 12 feet blow the bed of the stream.

The cost per mile for major bridges amounts to Rs. 1,326, whereas for minor bridges to Rs. 3,158.

Flow of water through pipes :—There being resistance to flow in the pipe, a portion of head of water is lost in overcoming friction.

At any portion of the pipe it is the net head which is lost producing the flow through the pipe.

Of the various formulæ used for computing the volocity of flow through the pipes, Kutter's, Bazin's, Hazen's, and William's are generally employed :—

The Hazen's and William's formula for flow is :

$$v = 1 \cdot 32\ cm^{0 \cdot 63}\ s^{0 \cdot 54}$$

where, m = hydraulic mean depth

$= \frac{\pi D^2}{4} \div \pi D$ or $\frac{D}{4}$ for a circular pipe D being its diameter.

s = slope

= *difference in head between the two ends of the pipe*

$= \frac{h}{\ }$ length of the pipe.

C = a constant having different values for different kinds of pipes.

$$\text{But, } Q = \frac{\pi D^2}{4} \times V$$

$$\therefore Q = \frac{1 \cdot 32\ C\ D^{2 \cdot 63} \times h^{0 \cdot 54}}{4^{1 \cdot 63} \times s^{0 \cdot 54}}$$

where, Q = discharge per second.

The list for the value of C :—

			C
Smooth wood-stave pipes	...	...	125
Unplaned wood flumes	...	...	112
Smooth concrete	...	...	130
Cast-iron pipes, new	...	...	130
Rivetted steel pipes, new	...	...	110
Rivetted steel pipes, old	...	...	100
Rough, tuberculated iron pipes	...	...	40 to 90

From the above exponential theorem :—

$$h = \frac{L\ F\ v^{1 \cdot 853}}{D^{1 \cdot 166}} \text{ ft.}$$

where, h = head lost in friction in feet.
L = length of the pipe in feet.
v = velocity in feet per second.
d = diameter of pipe in feet.
F = a constant.

Value of F for different materials :—

		F
Smooth wood stave pipes	...	0·000392
Unplaned wooden flumes	...	0·00048
Smooth concrete	...	0·000363
Cast-iron pipes, new	...	0·000363
Rivetted steel pipes, new	...	0·0005
Rivetted steel pipes, old	...	0·0006
Rough, tuberculated iron pipes	...	0·0032 to 0·00072

The formula for William and Hazen is, apparently, the most reliable, particularly for power-plant works, as the experiments to which these investigators had access included large pipes.

The Kutter's formula has been explained in the chapter of hydraulics, it can also be usefully employed in calculating the size of a pipe. Bazin's formula is rarely used.

Enger's Equation for steel pipes :—

For the economic size of pipe, Enger has derived an equation which is based on Adam's theory. The equation is as follows :—

$$d = 9{\cdot}08 \left\{ \frac{e\,b\,c\,Q^3}{a\,(R+i)\,t\,C^2\,(1+n)} \right\}^{1/6}$$

Where, d = most economic diameter in inches.

e = over-all efficiency of the plant to point of scale and expressed as a decimal.

b = value of lost energy, in mills per kilowatt-hour at the point of scale.

c = a coefficient for use in determining the productive head.

Q = average discharge in cubic feet per second.

a = the cost of steel in the pipe in rupees per pound.

R = the desired per cent. net return on money invested, expressed as a decimal.

i = the estimated annual operating tax and depreciation charges in per cent. of the construction cost expressed as a decimal.

t = the thickness of the pipe-plate in inches.

C = Chezy's friction coefficient and for rivetted steel pipes it is equal to 110.

n = per cent. over-weight due to laps, rivets, etc., expressed as a decimal.

In a particular design :—

e = 0·75	a = 0·10
b = 8·0 miles.	R = 0·15
c = 2·0	i = 0·02
Q = 200 sec.-ft.	t = 0·25
n = 0·2	C = 110

$$\therefore d = 9{\cdot}08 \left\{ \frac{0{\cdot}75 \times 8 \times 2 \times 200 \times 200 \times 200}{0{\cdot}1 \times 0{\cdot}17 \times 0{\cdot}25 \times 110 \times 110 \times 1{\cdot}2} \right\}^{1/6}$$

= 97 inches.

I. G. White & Co. uses another formula on the basis that a pipe-line is most economical when its size should be such that

the value of the power annually lost in friction *plus* the annual interest, profit and depreciation charges on the pipe-line should be a minimum. For the steel pipe the formula leads to :—

$$d = \left\{ \frac{320 \times 62 \cdot 4 \times x^2 \times q^3 \times e}{\pi^3 \times t \times m \times i \times C^2} \right\}^{1/6}$$

where, d = economic diameter in feet for thickness t.
t = thickness of the pipe in feet.
m = weight of material in pipe line in lbs. per cu. ft. = 490.
q = average flow of water through the pipe during 24 hours expressed in cubic feet per second.
e = sale value of 1 foot-pound per second for one year, measured in water before delivery to turbine.
i = annual interest, profit and depreciation charge on 1 pound of material in pipe-line in place, expressed as a ratio.
C = coefficient of friction used by in Hazen's and William's formula.

$$x = \sqrt{\frac{\text{Average of the cubes of load curve ordinates}}{\text{Cube of the average of load curve ordinates}}}$$

The value of x for a 50 per cent. load factor generally varies from 1·3 to 1·5.

Stresses in pipes :—The forces acting to distort or rupture pipes are :—

(1) Bursting due to internal pressure.
(2) Collapse due to formation of a partial vacuum in the pipe and consequent unbalanced external pressure.
(3) Breaking due to flexure stresses when the pipe is carried on supports spaced along the line.
(4) Rupture due to sliding on hill sides.
(5) Rupture due to contraction and expansion with the temperature changes.

Bursting of pipes :—The thickness of plate required to resist the bursting pressure is

$$t = \frac{p \times D}{2 \times S \times E}$$

where, t = thickness of metal in inches.
D = diameter of pipe in inches.
S = working stress in pounds per square inch.
E = efficiency of joint.

p = maximum pressure in pounds per square inch.
S = 15,000 for Siemens Martin steel plates.
E = 60 to 65 per cent. for single-rivetted joints.
= 70 to 75 per cent. for double-rivetted joints.
= 90 to 100 per cent. for welded steel pipe joints.

The calculated thickness should be increased by 1/16 inch to allow for corrosion.

As for example consider a 9-ft. diameter penstock with double-rivetted joints under a head of 200 ft.

$$\therefore \quad p = wh = \frac{62 \cdot 4 \times 200}{144} \text{ lbs. per square inch.}$$

$$E = 0 \cdot 7$$

$$\therefore t = \frac{62 \cdot 4 \times 200 \times 9 \times 12}{2 \times 144 \times 15000 \times 0 \cdot 7}$$

$$= 0 \cdot 222 \text{ inches.}$$

Allowing 1/16 inch for corrosion.

$$t = 0 \cdot 222 + \frac{1}{16} \text{ or } \frac{5}{16} \text{ inch, approximately.}$$

The provision is to be made to prevent shocks due to water hammer, penstocks are frequently subjected to stresses, which are greatly in excess of the normal stresses due to the static head. The usual methods of providing against this are three in number, *viz.*, relief valves, surge tanks and air chambers.

Collapse of Pipes :—Near the power-station steep incline occurs and there is a considerable length of pipe, say 80 ft., or more, extending back beyond the bend which the pipe makes intending downwards ; there is a danger of the collapse of the penstock unless it be designed to resist external pressure. It is due to the fact that the rapid opening of the water-wheel gates will allow that portion of the water column in the inclined portion of the penstock to accelerate rapidly. As the acceleration of the column of water in the steeped portion is greater than that of the same in the horizontal portion, the separation of the water at the bend will take place when vacuum will be formed in the pipe, and the external air pressure may cause it to collapse. In order to prevent collapse, *air valves* or *vent pipes* are to be provided at or near the bend where the penstock takes its steep, downward inclination.

No large thin-walled metal tube is truly circular in cross-section when installed, but varies more or less from true roundness. Even if it were exactly circular when constructed,

its own weight, and that of the contained water would cause it to flatten slightly and take an elliptical form, the vertical diameter being shorter than the horizontal diameter. This distortion makes the pipe easier to collapse, but without a definite known ratio between the major and minor axes of the flattened circle, no rational mathematical formula is possible.

The empirical formula used for safety against collapsing due to sudden drop in pressure is given by :—

$$P_1 - P_2 = \frac{5 \cdot 2 \times 10^5}{S} \times \left(\frac{t}{d}\right)^3$$

Where, P_1 = external pressure in lbs./sq. inch.
P_2 = internal pressure in lbs./sq. inch.
t = thickness of pipe wall in inches.
d = diameter of pipe in inches.
S = factor of safety against collapse of pipes.

For pipes buried in earth, a value of S = 5 should be used and for pipes on saddles, S = 10 is a safe value.

Whether the pipes are buried or not, they should be carried on concrete piers. Heavy anchorage blocks should be inserted at all the vertical and horizontal bends and where there is considerable variation in temperature expansion joints are to be provided to take care of expansions and contractions.

Freezing of water inside steel pipe :—The water inside the pipe line must be prevented from freezing. This generally takes place in case of very cold weather when the plant is shut down for any reason. It is necessary, then, to let the water flow through a by-pass, or other means, at such a velocity as will prevent freezing.

Wherever possible, in the case of very cold climates, the pipe should be buried in the ground or covering them with concrete housing.

There is practically no danger from freezing, the frost line is twice this distance below the surface, provided the centre line of the pipe is below the frost line.

Buried *vs.* Exposed Pipes :—The steel pipes should not be buried unless local conditions render it necessary.

The conditions favourable to the buried pipe are as follows :—

(*a*) Pipes extending down steep hill-sides on earth foundation make anchoring and supporting very difficult. If the pipe be buried, no anchoring and supporting will be necessary.

(*b*) On steep side-hill locations, there is frequently danger from land-slides, snow-slides and falling rocks, which would injure the pipe unless buried.

(*c*) For long pipes it is less expensive to bury them than to provide other means of protecting them from freezing.

(*d*) Where the pipe passes through an earth-cut, it is often cheaper to bury it with excavated earth than to provide cradles and sills.

(*e*) Expansion joints can be reduced or eliminated for buried pipes.

Conditions favourable to the exposed pipe are as follows :—

(*a*) More room is provided for construction.

(*b*) Exposed pipes are accessible for frequent inspection, maintenance and repairs.

(*c*) Buried pipes have shorter life as they cannot be inspected frequently.

(*d*) Exposed pipes are less expensive to install if the material is difficult to excavate.

Points to be considered in locating a pipe line :—

When deciding upon a pressure pipe line it is necessary to consider the following points :—

(1) Make sure that the hill-side is good.

(2) See that the pipe line is as straight as possible, *i.e.*, free from bends, sharp drops, etc.

(3) See that the pipe is given the best slope possible.

(4) See that it is securely anchored and well-backed, and that supports are properly spaced.

(5) Take every precaution as to its being safely supported.

(6) Ascertain that the downward pressure of the pipe line does not cause injurious pressure on the distributing pipe or to the power house structure or turbine.

(7) Make proper allowance for expansion, as great changes in temperature can cause leakage, loosen supports and anchorage, etc., and thus endanger the whole pipe line and the hill-side.

(8) See that all the pipes are supported and their foundation is good.

(9) Give the pipe line a good clear right-of-way and every protection, *i.e.*, see that it is protected from the direct rays of the sun.

Design of steel pipes :—The following points are to be considered in the design of a steel pipe :—

(*a*) It should be required to develop the scheme so as to yield the maximum amount of power irrespective of first cost, because of an unlimited power market.

(*b*) It is better to develop the scheme so that the annual cost of operation will be a minimum for a maximum output, and

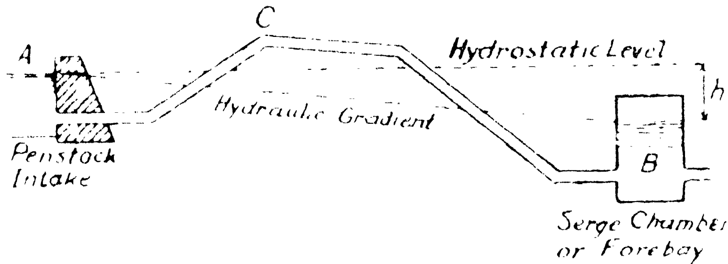
the annual net income a maximum but where the working head and water supply are ample and there is a power market.

Siphon :—Siphons are commonly used when a pipe line is laid overground which is higher than the hydraulic gradient. The flow of water in a siphon is calculated from the relation.

h = loss at entrance + friction loss + loss at exit.

$$\text{or, } h = \frac{v^2}{2g} + Ls + \frac{v_1^2}{2g}$$

Fig. 7.



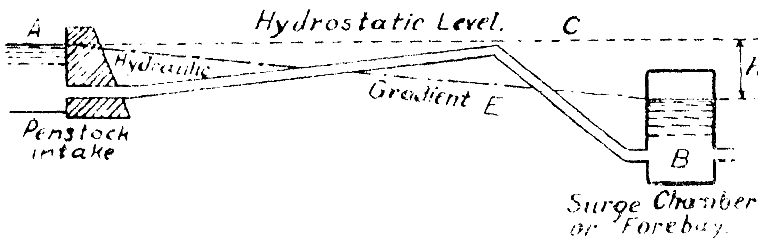
Pipe line above Hydrostatic level.

The height of the siphon above hydrostatic level should not exceed 34 feet theoretically (at sea level). In practice it should be less than the barometric height by an amount h.

$$h = \frac{v^2}{2g} + L_1 s + \frac{v_1^2}{2g} \quad \ldots \quad \ldots \quad (1)$$

where L_1 is the length of the pipe line from A to C or to the highest point of the siphon.

Fig 8.

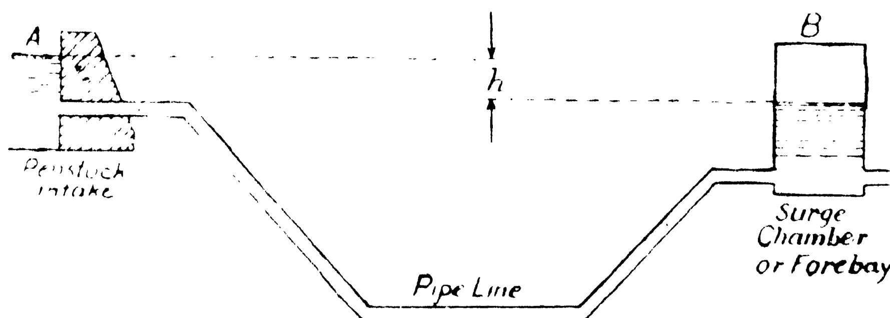


Pipe line below Hydrostatic level

At the summit it is common practice to locate an air valve because the air accumulates there during the working of the pipe system; on the other hand, a vacuum could be produced on closing the penstock intake, in which case the pipe may collapse unless provision is made for the admission of air.

Naturally, the pipe is initially filled by means of a pump or other system, in order to start the flow. In the second case shown in the figure it is not necessary to charge the siphon, because C is below the hydrostatic level and less than the barometric pressure by a head equal to CE. However, air troubles must be expected unless air valves are provided.

Fig. 9.



Inverted Siphon.

A case frequently met with is that of the reversed or inverted siphon, this being a practical method to cross valleys.

In the above figures, A is the penstock intake and B is a chamber or stand pipe to receive the effects of the swelling harmonic waves produced by the sudden shutdown at the power house.

Equation (1) given above is applicable in all the above three cases.

Wind Pressure :—The penstocks and other conduits must be sufficiently strong to resist the wind pressure on the supported pipe According to Mr. T. M. Gilmar—(Engineering Record).

Let the unit pressure be p.

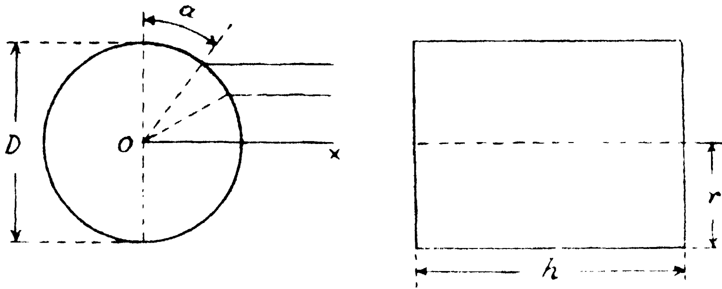
Then the pressure on any small element $h\,r\,d\alpha$ will be $(h\,r\,d\alpha)\,p \sin^2\alpha$.

The component of the normal pressure parallel to OX is $h\,r\,d\alpha\,p \sin^3\alpha$. (Fig. 10).

The total pressure in the direction OX is

$$P = 2\,h\,r\,p \int_0^{\pi/2} \sin^3\alpha \cdot d\alpha = \tfrac{4}{3}\,h\,r\,p.$$

Fig. 13



This is the pressure on the cylindrical surface in the direction of O X in terms of the radius and the unit pressure. The total pressure p′ on a plane surface of the same length and of a width equal to the diameter of the pipe.

$$P' = 2\,h\,r\,p.$$

$$\therefore\quad P = \tfrac{2}{3}\,p'.$$

Or, the pressure on the cylindrical surface is two-thirds that on its diametrical projection.

Water Hammer :—At the time of filling in, or by suddenly closing the gate of discharge, penstocks of hydro-electric plants are often subject to a rise in water pressure due to the kinetic energy of moving water being brought to a sudden stop. This phenomenon is called water hammer in pipes, and results in increasing the stresses in the shell of the penstock. This effect is remedied by surge pipes usually installed along the pipe line, the area of such surge pipes being at least equal to the area of the penstock.

The common formula to calculate the rise in pressure in a penstock, due to closing of gate, is :—

$$p = 0{\cdot}0134\,L\,\frac{(v_1 - v_2)}{T}$$

in which, p = rise in pressure in lbs. per sq. inch.

v_1 = normal velocity of water in penstock in ft. sec.

v_2 = velocity after change in gate opening in ft. sec.

L = length of penstock in ft.

T = time in seconds required to change gate openings.

The coefficient 0·0134 is rather small, and Mr. G. M. Peck (Engineering News, Aug. 11, 1910) proposes that its value be

taken at 0·0201, the latter being safer. Mr. Merriman proposes the formula—

$$p = 0{\cdot}027 \frac{L v}{T}$$

where, p, L and T are as before and v = velocity of water in ft. per sec.

Sudden variations in the velocity of water in pipes may produce a positive water hammer effect or a negative one, according to the direction of the variation of the velocity. These phenomena may occur consecutively in the same pipe line, and whereas the positive water hammer may sometime assume dangerous values, the negative water hammer cannot cause injury, because if vent pipes are provided, the pressure inside the pipe cannot become less than the atmospheric pressure.

According to Mr. L. Allievi (Revu de Mecanique, Paris, January-March, 1904), the formula for negative water hammer, in order to determine the time T of opening so that the maximum decrease in pressure shall not exceed a given time (1 – s), is

$$T = 2 \frac{L v' \sqrt{s}}{g H_0 (1 - s)}$$

in which, T and L are as before, L being in meters.

g = acceleration due to gravity = 981 cm./sec.

H_0 = head of water in meters.

v' = velocity of water in pipe, in meters per sec.

$s = \frac{H}{H_0}$, where H is the resulting pressure in meters.

Calculation of Surge Tank

Let M = mass of water flowing in the pipe of length L.

V_c = velocity of water in feet per second.

The energy of the flowing water will be M $V^2/2$.

Now suppose the gate is suddenly closed, the water in the surge pipe will be raised to such a level as to counterbalance the energy of the flowing water.

Let S_1 = the height of water in the surge tank, when V = 0.

$\therefore$ The equation of equilibrium is $\frac{MV^2}{2} = \frac{w S_1}{2}$

where, w is the weight of water raised through $\frac{h}{2}$ feet and it becomes $62{\cdot}5 \text{ Ac } L V_c^2/2g = 62{\cdot}5 \text{ As } S_1 \frac{S_1}{2}$

If the area of the pipe is called A_c and that of the surge tank A_s the value of h is then found to be –

$$S_1 = Vc\sqrt{\frac{L A_c}{g. A_s}}$$, in which g = the constant of gravitation.

If the ratio $\frac{A_c}{A_s}$ is expressed by m, the above formula becomes $S_1 = V\sqrt{\frac{mL}{g}}$.

Supposing the average thickness of the plate is equal to the thickness of the bottom, the most economical tank of definite volume is that, the height of which is equal to the diameter, when the top is closed; or its height is equal to the radius, when the top is open.

Let H = height of the tank, r = its radius.

Its plate area is $A = 2\pi r^2 + 2\pi r h$.

Differentiating and equating to zero –

$$4\pi r + 2\pi H = 0$$

Solving for H, $H = 2r$ = diameter.

When the top is open $A = \pi r^2 + 2\pi r h$.

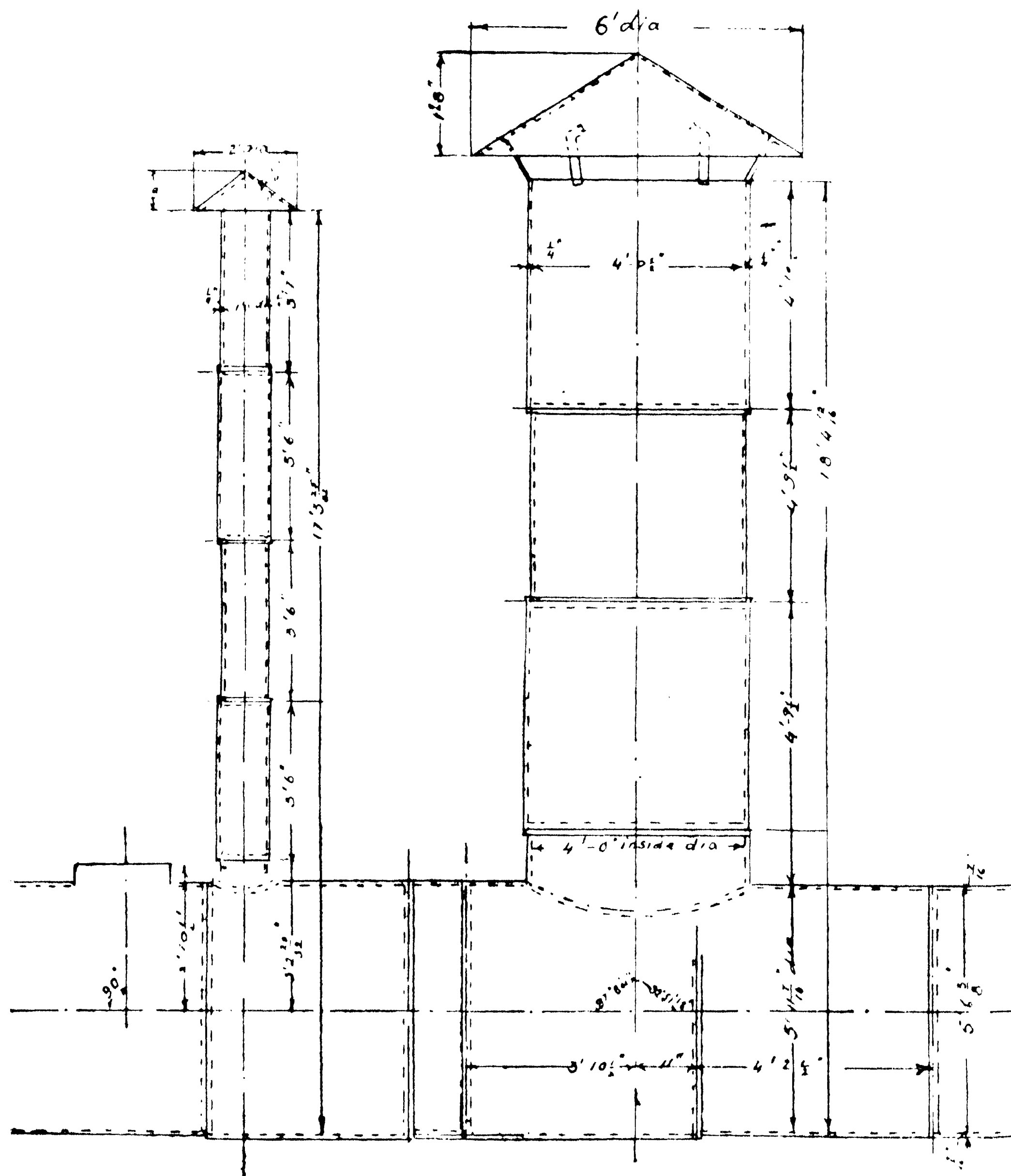
Differentiating and equating to zero—

$$2\pi r + 2\pi H = 0$$

$$\therefore H = r = \text{radius}.$$

The general formula for the height of surge on instantaneous closure was $S_1 = V_c\sqrt{\frac{L}{g} \times \frac{Ac}{As}}$, where S_1 denotes the height of the surge in feet, L the length of the conduit in feet, V_c the velocity of the water in the conduit in feet per second and A_c and A_s the areas in square feet of the conduit and surge chamber, respectively. In the formula no account was taken of the loss due to the change of direction of flow or of eddying due to change of section, so that a constant c, less than unity, should be inserted before V_c. The value of that constant could not be determined otherwise graphically, as it would vary with the velocity V_c. Hence, for a fixed value of c, S_1 would not vary with c V_c, but with c V_c^2 Sir Michael Nethersole, when investigating the problem of the surge chamber of the Tata Power Company, had tentatively adopted a constant c = 0·065 for instantaneous closure. But clearly the closure must,

Fig. 11.



Surge tower at Shiva Samudram.

in practice, always be gradual, and Sir Michael concluded that it would be safe to assume $S_r = 0 \cdot 8 S_1$. The final formula then became—

$$S_r = 0 \cdot 05 \, Vc \sqrt{\frac{L}{32 \cdot 2} \times \frac{Ac}{As}}$$

From which the value of A might be calculated from $A_s = 0 \cdot 000084 \ V_c^4 \times L \ Ac/S_r^2 r$, or if the surge was to be confined to the level of the reservoir behind the flume, then h_c^2 might be substituted for S_r^2.

Material at Pykara :—Lap-welded steel tubes are used for the upper section and seamless tubes of greater thickness are used for the lower sections. In the manufacture of the seamless steel tubes solid round billets are used, which, after being heated to the required temperature, are pierced in special piercing mills of complicated mechanism. The hollow billets thus obtained are rolled by a pair of rolls of special profile over a mandrel. These pipes are admirably suited for high pressure and are used for the bottom section.

For steel tubes lap-welded by the water gas process, an easily weldable open hearth steel having a tensile strength of 21·6 to 28·5 tons sq. in. and a minimum elongation of 20 % is used. The large elongation of the material renders it especially suitable for turbine pipe lines.

In fixing the thickness of the plates it is necessary to consider not only the pressure due to the net head but also the additional pressure caused by water hammer. Another important consideration is the safety from collapsing due to sudden drop in pressure. A study must be made of the entire penstock from the headgate to the turbine casing and the exact drop in pressure calculated at each section under the most severe conditions. Such conditions might occur when a turbine is running light, and a short circuit occurs on the generator, in which case the turbine gates open wide very quickly, and there is a tendency to accelerate water in various sections of the pipe.

In a long penstock, as in Pykara, the water column below a certain section may have sufficient head to be accelerated more rapidly than the column above. This may cause a break in the water column at the section in question and a considerable vacuum which is likely to collapse the pipe. So it is expedient to provide against any excess of external, over internal pressure, at any point in the pipe iine.

Mr. J. W. Mears, pp. 260 and 261—Discussion on Hydro-Electric Power Development - Excerpt Minutes of the Proceedings of the Inst. of Civil Engineers, V. 228, Session 1928-1929. Part 2.

The working pressure specified for the pipe is that due to static head + 12½ % of the head for instantaneous gate closure within the critical closure time. So the most severe conditions have been presupposed. Gates never close instantaneously, but the critical closure time for long penstocks is comparatively high. For these lines it is about 4 secs. As relief valves will be installed, the pressure rise will be negligible ; and should a valve fail to act, the maximum possible rise is well within the factor of safety. The factor of safety is 4 times the working pressure corresponding to ultimate stress of steel or 25 times static head + water hammer, whichever is greater.

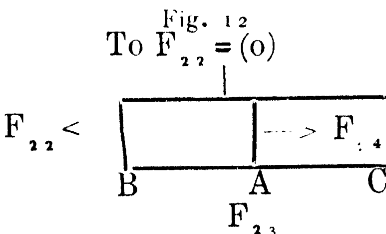
The pipe lines—buried or not—should be carried on piers of concrete. Heavy anchorage blocks should be inserted at all vertical and horizontal bends and where there are considerable temperature variations. Expansion joints should be provided to take care of the expansion and contraction of the pipe. While the stress may be well within the elastic limit of the pipe material and would have little influence on the pipe itself, the thrust caused by the expansion may throw a very high stress on the anchorages. Pipe lines filled with expansion joints are only subject to a negligible axial stress, while, on the contrary, pipe lines without expansion joints must take up this strain in full. By providing expansion joints, therefore, a material saving can often be effected in the cost of anchorage blocks and piers, especially where their construction involves difficulties owing to the steepness of the grade and lack of handling facilities.

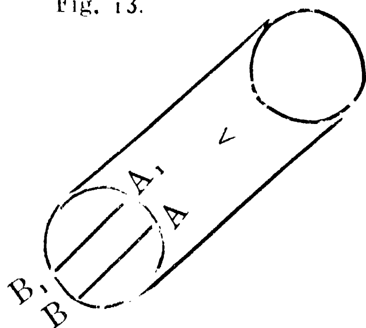
Erection :—As already remarked, the route is first fixed after elaborate investigations and the pipes are manufactured with reference to the route and other hydraulic and survey data. The pipes are all numbered to enable the transport gang to know where exactly to haul down a pipe and the erector to know where exactly to fit it. In each pipe an arrow mark is painted conspicuously to enable the erector to know which side of the pipe—or rather face—should be up. Also in the faces of the anchor point pipes there will be small punch marks which should be in a horizontal level when the pipes are filled. The welded seam of the pipes are brought in the central plane of the pipes to reduce bending stress there. The pipes are painted with special paints externally and internally and the faces protected with planks of wood bolted to the collars. If in transport or otherwise the edges are spoiled, the damaged portion is welded by an oxy-aceteline plant and hammered out and the surface made

true. The pipes are hauled down from the haulage truck on to the proposed route of the penstocks as closely as possible to the place where they should be.

In truly aligning the pipes much time will be spent in laying the anchor pipes in position. Once the two anchor pipes are laid at an anchor point corresponding to the two elevations of the line on either side of the point, pipes have only to be connected together and all the pipes sighted to be in the same elevation.

Preliminary to the laying of the pipes a scantling is fixed at every anchor point on two wooden posts at an elevation of exactly 3 ft. or any other convenient but constant distance above the centre line of the pipe line (proposed). Also fine red lead marks are given on the scantlings to mark the vertical axis of the centre profile of the pipe line. This survey, being the basis for further work on the penstock erection, is very accurately carried out with sensitive theodolites. The two anchor pipes for the upper and lower elevations of the anchor point are first laid in position on eye judgment and they are bolted together. The theodolite is then fixed on the scantling with the vertical axis in line with the vertical red lead mark on it. A finger point with a small conspicuous mark on it is fixed on to the next scantling (say F_{22} if the anchor under erector is F_{33}) with the finger in line with the red lead mark. The distance of the finger point from the scantling level is the same as the centre of the telescope above the scantling on F_{23}, so that the line of sight is parallel to the proposed route of the penstock line. The observer now proceeds to truly align the vertical plane of the pipes. All along, it will be remembered, only one side, *i.e.*, F_{23} to F_{22}, is to be aligned. A spirit level is fixed to the face B and the true central or rather horizontal plane being fixed, a plumb is passed through the bubble point of the level and the vertical axis of the pipe thus marked. A rule is now fixed on to this axis and the observer sees whether it is in his line of sight. The rule is slided or better a plumb is moved along the face until it is in the same line of sight or rather the vertical axis of the telescope. AB is the true central axis of the pipe and A_1B_1 is this new axis found. That is to say that AB should be in the position A_1B_1. So the pipe is displaced by applying crowbars to this position and the whole process again repeated until AB

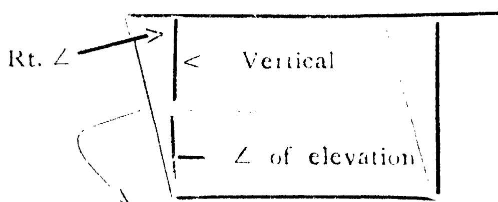




and A_1B_1 are identical. This is not all. The axis of the line of sight may not be the same as the axis of the pipe but still they may appear to be the same if they intersect at the face of the pipe, *i.e.*, at AB. Therefore, in final test the rule is fixed to AB and an angle of steel fixed to it at right angles to it and slided up or down, and the two axes are thus verified to coincide if the angle and the line of sight coincide.

Another thing to fix is that the pipe is laid in true elevation. This elevation is to be the elevation of all pipes from F_{23} to F_{22}, so that considerable care is taken here. The vertical height above the correct centre of the pipe to the scantling level is a known constant. The distance from the scantling level to the telescope centre, of course, vertical, is also a fixed constant. Thus the vertical distance from the centre of the pipe to the centre of the telescope being known and the elevation of the pipe also being given, as how much it should be, the slanting height measured along the face of the pipe (or to its centre line) from the centre of the pipe to the line of sight is easily computed from this.

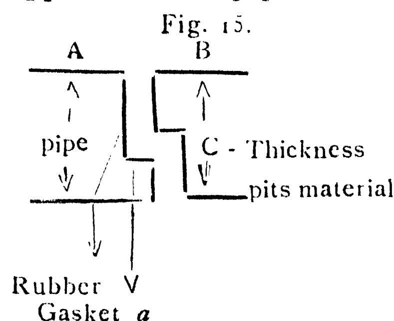
The rule is now fixed to the central axis of the pipe with a mark on it to show where exactly the line of sight of the telescope should cut it (of course the point being marked from the above data). If the line of sight cuts it *above* this mark, it means that the mark has to go so much high, which, in turn, means that the pipe has to be raised so much. This is done by raising it up by screw jacks and giving packings underneath. These two processes are repeated several times until finally both the "Elevation" and "Centre line" are true. The erector then tries to fix the upper section, *i.e.*, the pipe from F_{23} to F_{24}. This also being laid as detailed above, he goes back to the other pipes and finally sees whether both are in alignment. Generally, when one pipe is disturbed, the other two being bolted together is slightly disturbed, so that the process of true alignment is an elaborate one requiring much skill and a good comprehension of labour.



Slanting height = vertical At × cos ∠ of elevation

For final adjustments when the pipe has to be lifted without later shifting, the hydraulic jack is used. In raising or lowering the pipes, they can be anchored to some strong place by means of wire ropes through "chain tackles" and the pipes can be raised up or lowered by working this tackle. When the distance to be passed through is longer, the pipes may be connected to a hand-operated or an electric-driven drum wound winch and the pipes raised or lowered by operating the winch. In every case skilled coolies with crow-bars would be keeping the pipe in position and packings of wood and wedges would *always* be put under the pipe, the pipes never being allowed to slide on ground. When the pipe is laid in true alignment by putting the wooden and iron plate packings underneath—thin iron plates for final adjustment—and the saddles are completed when the packings could be removed. At the anchor points, where the tendency of the pipe will be to be lifted up by the pressure of the water, there are what are called "building down bolts." Pits of about 2 or 3 feet will be provided in the concrete seating of the anchors for the bolts. When the bolts are put over the pipe and tightened (2 bolts per anchor), they first depend for their grip on the friction between the bolts and the ground of the pits and afterwards when the pits will all be filled in by concrete, the bolts can be tightened and the grip will be very good. Afterwards the pipes will be covered up with concrete and the anchor would resist any pressure of the water. To get a better grip into the concrete these pipes are provided with thin flaps which would go into the concrete. (*Vide* Fig. 70, p. 286).

The *jointing* of the pipes is of prime importance in erection. The two faces to be lapped together must be cleaned with emery paper and dry cloth just before they are to be jointed. The joints are somewhat of this shape. Before jointing the pipes a rubber ring (not endless) of less than $\frac{1}{4}''$ dia. is fixed round the pipe A in the position marked "a". It will be noticed that the upper end of pipe A and the lower face of B are shown here. Any twist is avoided. When the pipe "B" is lowered down, the portion C of it laps over "a" of A and the ring is jammed in between the two faces of A and B. This forms a watertight fit and the pressure of water only tends to make the rubber ring more watertight. At the anchor points a second ring is fixed in position "K" from inside. When the two pipes have been connected and bolted together, one man goes into the pipe



with the ring and presses it into position K cutting the ring, stretching it while pressing and final jointing the ring by solution. This prevents leakage to a further extent and ensures that the joint is tighter with increasing pressure of water inside.

It will be observed from the above details of erection that an anchor point has first to be fixed and then pipe erected *up* from the point (refer fitting of rings) to the next *higher* anchor. It was said that the distance between two anchor points would be truly estimated and the pipes manufactured accordingly. But after manufacture always some discrepancy of a few millimeters creep in necessitating a difference between "theoretical" and "actual" length of pipes. This discrepancy is accommodated for by the expansion joint which is fitted at the end of the bottom pipe (*i.e.*, for lower elevation) of every anchor point. Besides this, these joints take care of expansion and contraction in the lengths of the pipes due to temperature variations. The stress may not damage the pipe, but it would enforce heavy duty on the anchorages. The other functions of expansion joints were already mentioned

The true alignment of the connected pipes is a comparatively easy affair. The rule can be fixed to every face and elevation fixed. The centre point, too, could be fixed as before, or by using an angle of steel on the longer side.

Anchors

Anchorages are located at each vertical or horizontal bend and these are simple gravity structures of reinforced concrete.

The anchoring at several points varies according to the pressure to be resisted. Since there is to be an anchor at every bend, it can easily be seen that the pipe line from one anchor to the next point is in a vertical plane having a definite elevation.

The bends of the pipe line, either vertical or horizontal, are securely bedded in a concrete anchor block, which has been designed to withstand the following forces :—

1. Water pressure due to the static head.
2. Weight of the pipe line between two anchors is borne on the lower anchor.
3. Frictional force due to the expansion and contraction of the pipe line.
4. Weight of the anchor itself. Thus, there are four types of anchor blocks designed.

(1) Anchor Blocks on Convexity.
(2) ,, ,, ,, Concavity of bend.
(3) ,, ,, ,, Horizontal bends.
(4) Intermediate anchor blocks which take the expansion thrust, are located in such a way that the distance between the two successive anchors does not exceed 300 ft.

Fig. 16.

The anchorages should be so designed that the line of action of all the forces acting above the base shall fall within the middle third of the base. As shown in the figure, we have, in addition to the weight W of the anchor block, the force due to the pressure of water in the pipe—P, weight of pipe from X to Y in pounds—A, the weight of water in the pipe—B, and D, the resistance to sliding due to friction of supports S in pounds (= ·5 C). These frictional forces set up on the intervening saddle are caused by expansion and contraction consequent on the changes of temperature.

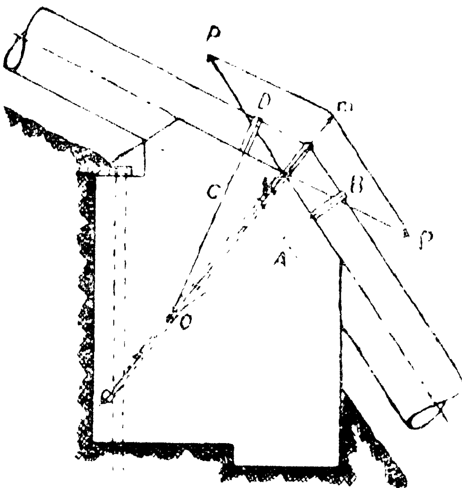
Now K is the point at which the resultant force may be said to act and this should be within the middle third of the base. To transfer these forces to the concrete block, angle flanges are riveted or welded to the bend. Sometimes, for the even distribution of pressure, cast iron collars are provided and these must be anchored down to the block, if the upward forces are considerable.

Sometimes, when long straight portions of pipe occur, intermediate anchorages, which are, of course, lighter than the main anchorages, are provided, at distances of 200 yards or less.

These are necessary, primarily to reduce the forces acting on the main anchorages and secondarily to reduce the length expanding and contracting as one unit.

Elbow :—The question of stability of an elbow in a pipe line is a serious problem for hydraulic engineers. The forces of an elbow tend to pull it out from its anchorage thus producing dangerous strains in the pipe itself if not properly secured. In a static condition, when water is not flowing, the faces of the elbow, Fig.16 AB and CD, receive normal pressure, which are transmitted integrally to the section D b, so to compute the pressure which is exerting itself on the shell at that point, it is necessary to find the resultant of the two normal pressures.

Fig. 17.



These pressures are each equal to P = 62·5 A H.

where, A = the area of the section of the pipe

H = the static head.

The resultant, is therefore, $R = 125\ A\ H \cos \frac{x}{2}$.

This relation is immediately observed in the figure, where the isosceles triangle KPM gives KM = R = 2 KP cos PKM.

There is also the centrifugal force to be added when the water is flowing, given by the relation

$$F = \frac{M v^2}{R} = \frac{w}{g} \times \frac{v^2}{R}$$

where, M = the mass of moving water between the faces AB and CD.

v = its velocity.

R = radius of curvature of the elbow.

w = weight of the water of mass M.

g = acceleration due to gravity = 32·2.

Hence, the elbow must be solidly bolted to a pier which will offer resistance by its own weight. The resultant of the masonry and water forces should intersect the base of the pier within the middle third to provide a safe factor against overturning. In special cases of sharp turns, the pier itself must be strongly anchored, as shown in the figure (*vide Fig. 70, p. 286*).

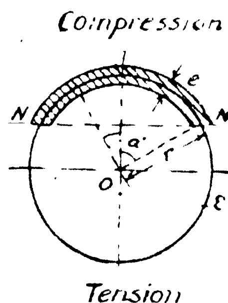
Pipe Supports

Each pipe length is provided with one concrete supporting block (shown at S in the previous figure). The pipe rests on a metal saddle. The friction coefficient between the two surfaces does not exceed 0·5. So the changes in temperature will cause the pipe to expand or contract longitudinally because of the pressure or expansion. The radius of the saddle is 1% more than that of the pipe so as to prevent distortion in excess while the pressure is being built up when filling. The pipe is allowed to be distorted somewhat and this relieves the saddle on some load. If the distortion of the lower part of the pipe is restricted by the saddles, when the pipe is being filled, while the upper part is comparatively free to undergo distortion (to expand freely), when under full-load and the saddle takes some of the stresses. This accounts for the cracks of the concrete.

Spacing of Piers for Reinforced Concrete Pipes :—The steel reinforcement is composed of circular and longitudinal bars. The former are provided to resist the water pressure and such pressure as may cause deformation (fill, etc.), while the latter

must take care of the bending effort of the pipe between supports. These longitudinal bars, uniformly spaced, will be replaced by another fictitious pipe, the steel shell of which shall have an equivalent area.

Fig 18.



Let—

r = mean radius of pipe.

e = thickness of concrete shell.

t = thickness of shell of equivalent fictitious pipe.

NN' = neutral axis defined by angle α.

α = angle limited by radius ON and the plane of bending.

m = ratio of coefficients of elasticity of steel and concrete.

Tension shall not exist in concrete, so that it does not have to be considered below the neutral axis. The position of the neutral axis is determined by the expression—

$$\tan\alpha - \alpha = m\pi t/e \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

Which gives the value of α.

The moment of inertia with respect to the neutral axis just determined is—

$$I = 2r^3 m t \left[\pi/2 + \pi \cos^2\alpha\right] + 2r^3 e \left(\alpha/2 - \tfrac{3}{4}\sin 2\alpha + \alpha\cos^2\alpha\right) \quad (2)$$

The maximum elastic stresses are then found by the formulæ in (M_x = maximum bending moment).

$$R_{tens} = \frac{m M_x r (1 + \cos\alpha)}{I} \quad \ldots \quad \ldots \quad (3)$$

$$R_{comp.} = \frac{m M_x r (1 - \cos\alpha)}{I} \quad \ldots \quad \ldots \quad (4)$$

From these formulæ—

$$\frac{R_{concrete}}{R_{tensile}} = \frac{1 - \cos\alpha}{m(1 + \cos\alpha)}$$

This gives α, knowing $R_{tens.}$ $R_{concr.}$

This value of α, substituted in equation (1), gives the thickness t, or, which is the same thing, the percentage of reinforcement.

The above formulæ have been develoved by P. Canfourier, and are very useful for practical purposes.

Numerical Example for Reinforced Concrete Pipe

Example :—To design a reinforced concrete pipe of 6 feet inside diameter, maximum hydrostatic pressure including the

water hammer effect is 60 ft. Pipe will be laid in wet sandy soil, and covered with a 4 ft. fill over the top. It will be supported on piers across a declivity at intervals to be determined.

There will be two layers of reinforcement, one near the outer and one near the inner faces of the pipe.

Assume that the thickness of the concrete shell is 8 inches ; the steel alone must resist the pressure of the water, the limiting stress being 12,500 lbs. per sq. in.

The thickness of the shell of an equivalent full steel pipe would be, according to formula (1) above—

$$t = \frac{60 \times 6}{4{,}800 \times 0{\cdot}75} = 0{\cdot}1$$

which corresponds to 1·2 sq. ins. of steel per lineal foot of pipe.

Stress due to fill.—The pressure due to fill is considered as being uniform and average depth of fill $7\frac{1}{3}$ feet.

$w_v = 100 \times 7{\cdot}33 = 733$ lbs. per sq. foot.

The intensity of horizontal pressure being taken as one-third of the vertical, and approximate depth in the centre line being $4 + 3\frac{1}{3} = 7\frac{1}{3} = 7{\cdot}33$

$$w_h = \frac{100 \times 7{\cdot}33}{3} = 244$$

By substitution—

$$M_b = -M_a = \left(1 - \frac{w_h}{w_v}\right) w \frac{r^2}{4} = \left(1 - \frac{224}{733}\right) \times 733 \times \frac{9}{4} = 1{,}138 \text{ ft. lbs.} = 13{,}656 \text{ in. lbs.}$$

The area of reinforcement will be determined by the formula—

$$M = sq \times 0{\cdot}86\, d$$

where s is the unit stress of steel.

$$q = \frac{M}{s \times 0{\cdot}86 d}$$

Assuming d = 6″

$$q = \frac{13{,}656}{12{,}500 \times {\cdot}86 \times 6} = 0{\cdot}2116 \text{ sq. in.}$$

This amount of steel is required along both faces. Having determined the amount of steel that will resist the water pressure, and the amount that will resist the stress due to fill, the total amount required will be = $1{\cdot}2 + (2 \times 0{\cdot}2116) = 1{\cdot}6232$ sq. in.

It will be necessary then to use $\frac{7}{8}''$ round bars, spaced 4 inches centre to centre.

As this same pipe is supposed to cross a declivity of the ground and be supported by piers, it is necessary to apply the formula developed by Canfourier for determining the proper spacing. The thickness, t, of an equivalent fictitious shell is 0·1. (It is assumed that the longitudinal reinforcement is of the same area as the transverse reinforcement). The thickness, e, of the concrete shell is 8 ins. The position of the neutral axis will be determined by proper substitution in formula (1), page 426. It is found that the value of α that satisfies the equation is approximately 50 degrees 20 mins., for which the arc value is 0·878 and tan α is 1·206.

In the formula—

$$\frac{R_{concrete}}{R_{tension}} = \frac{1 - \cos\alpha}{m\,(1 + \cos\alpha)}$$

Let $R_{tension}$ = 12,500 lb. per sq. in.

$$m = \frac{290{,}000{,}000}{2{,}400{,}000} = 12 \text{ (approximately).}$$

Solving for $R_{concrete}$ a value is found of approximately 230 lbs. per square inch.

Remembering that—

$$t = 0{\cdot}1 \qquad e = 8''$$
$$\alpha = 0{\cdot}878 \qquad \cos^2\alpha = 0{\cdot}407$$
$$\sin 2\alpha = 0{\cdot}983 \qquad r^3 = (2{\cdot}75 \times 12)$$

The moment of inertia will be obtained by formula (2), on page 426.

$$I = 2\,(3{\cdot}33 \times 12)^3 \times 12 \times 0{\cdot}1 \left[\frac{\pi}{2} + \pi \times 0{\cdot}407\right]$$

$$+ 2\,(3{\cdot}33 \times 12)^4 \times 8\,(0{\cdot}439 - 0{\cdot}737 + 0{\cdot}358) = 497{,}540 \text{ in}^4.$$

It is assumed that the pipe is acted upon as a uniformly-loaded beam with fixed ends. The maximum bending moment is given by—

$$M_x = \frac{WL}{12}$$

Where W is taken as 5,000 lbs. per linear foot (weight of pipe shell and water within).

Then $M_x = 5{,}000\ L^2$ in lbs. (L being in feet). Substituting this value of M_x in formula (3) and solving for L = 137 feet.

Knowing the safe span, the compressive stress for steel will be found by proper substitution in formula (4).

In the present case the compression is found to be 2,720 lbs. per square in.

Thrust Collars

At each bend anchor the external (resultant) forces up and down the pipe are transferred by the thrust collars fitted at side of every bend. They are strong enough to bear the axial thrust on the side.

Pipe Joints

Three main types of joints are provided :—

(*a*) The 55″ diameter pipes are connected by double-riveted lap joints designed to bear the forces of expansion and gravity of amount not exceeding 80 tons.

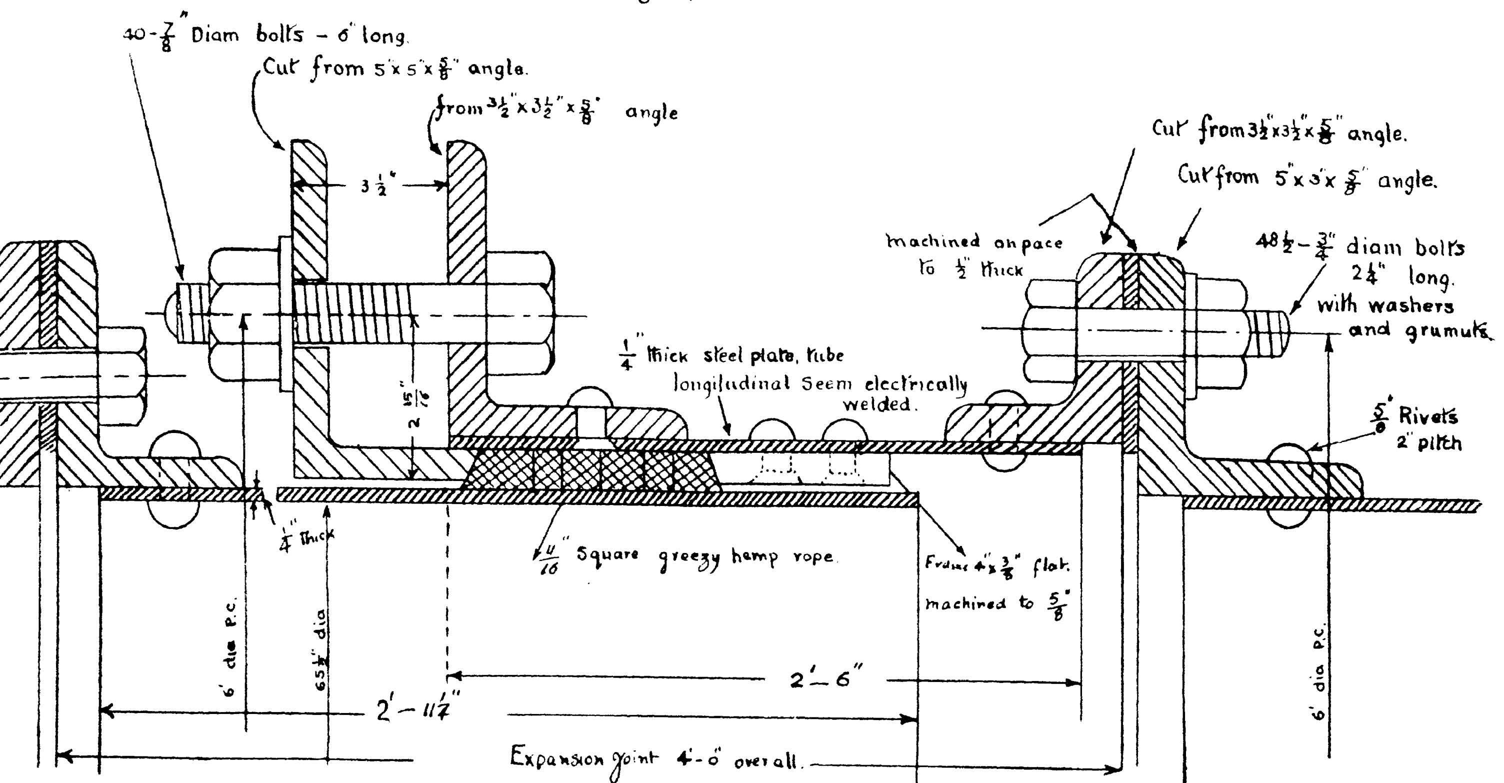
(*b*) The 38½″ diameter pipes connected up by means of riveted band joints designed to withstand the gravity and expansion pressure not exceeding 55 tons.

(*c*) All pipes of smaller diameter than 38½″ are connected up by means of *welded flange joints* to bear the expansion and gravity forces not exceeding 60 tons.

Expansion Joints

The expansion joints are located midway between anchors, so as to obtain most favourable balance of forces on anchors. Expansion joints are of welded pipes in the case of 4′ to 7′ pipes, and of cast steel in all lesser diameters. The travel, due to expansion and contraction between maximum and minimum temperatures, does not exceed 8″ in the most extreme cases. The packing material is of square-braided lump impregnated with graphite.

Fig. 19.



Expansion Joint.

Lap-Welded Steel Hydraulic Pipes:—The steel used in the lap-welded pipes has a tensile strength of 48,500-58,000 lbs./sq. in., a yield point of 28,000 lbs. per sq. in., an elastic limit of at least 26,000 lbs. per sq. inch and a minimum elongation in 8 ins. of 25 per cent. The steel contains not more than 0·15 per cent. of carbon, 0·06 per cent. of phosphorus, 0·05 per cent. of sulphur, and contains about 0·60 per cent. of manganese. The "yield point" and "elastic limit" are taken as defined in B.E.S.A. specification No. 56-1911. In none of the cases the yield point of the steel employed is less than the specification.

The maximum static head is taken as 2,011 ft. The maximum fibre stress in the steel does not exceed 0·60 of the yield point, and the efficiency of the lap-welded joints is taken as 90 per cent.

The net allowable maximum stress in the lap-welded plates, after making the allowances for 0·60 of the yield point and for the assumed 90 per cent. efficiency of the weld, based on steel with a yield point of 28,000 lbs. per sq. inch, is 14,500 lbs. per sq. inch.

An additional $\frac{1}{8}$ inch in thickness, which is not taken into consideration in stress calculations, is added to all calculated plate thicknesses to allow for corrosion. No thickness of welded pipe is greater than $1\frac{1}{4}$ inches, but this $\frac{1}{8}$ inch is allowed for corrosion, so that for the purposes of strength calculations the maximum thickness is taken as $1\frac{1}{8}$ inches. The thickness of the pipe is calculated after allowing an increase in the static head of 10 per cent. for water hammer.

Each pipe joint is flanged and provided with rubber ring packing registered in a suitable groove.

Cast steel Hydraulic Pipes, Bends, Tees and Valves:—Cast steel parts of British manufacture are in accordance with grade "A" or grade "B" of British standard specification, but the bodies of all valves up to and including 6″ bore are made of B.E.S.A. grade "A" cast steel. Test specimens were made and test carried out in accordance with the same specification.

Cast steel parts not of British manufacture have a tensile strength of 64,000/85,000 lbs. per sq. in., a yield point of at least 28,000 lbs. per sq. inch and a minimum elongation in 8″ of not less than 18 per cent. The steel does not contain more than 0·60 per cent. of phosphorus, 0·06 per cent. of sulphur, and contains about 0·50 to 0·65 per cent. of manganese.

Marking and Painting of Hydraulic Pipe Work and Valves:—Each straight pipe, bend tee-piece and valve is stamped with its test pressure and also with its identification number so

Item 13.

10% extra jointing material and bolts as spare.

Item. 14.

Suitable quantity of pipe dressing for repairs after erection.

The above to be generally in accordance with my Specification *No. 2900* of March 9th last and Messrs. Stewarts & Lloyds' Specification *No. 3089* of May 8th last, unless otherwise modified by particulars given below and on enclosed drawings.

Price:—C. I. F. Calcutta including insurance to destination and subject to slight modification to meet revised conditions—£ 4,416-0-0

Customer's Drawings:—Accompanying Drawings Nos. 90/4/A and 90/4/B give particulars of angles, lengths, etc., of the various sections and proposed arrangements of pipes and fittings at intake, surge tank and power-house.

The drawings are more or less to scale, but the scale has not been stated as it is not intended that any dimensions not given in figures should be taken or considered to binding.

Intake :- The arrangement here is quite simple and requires no explanation.

Surge Tank :—The original arrangement which provided for interconnection of L. P. and H. P. pipes through the surge tank has been modified, the L. P pipe now running straight into the H. P. pipe, as you will see from the drawing.

In Drawing No. 90/4/A the survey line from points 10 to 11 is that shown passing through the centre of the flange at the upstream end of the vertical bend adjoining the butterfly valve. The centre line of the H. P. pipe does not coincide with the survey line for obvious reasons, but will meet it at point 10. The angle, therefore, which the centre line of the pipe line makes with the horizontal, will not be 27 degs.-37 mins., as shown for the survey line, but slightly greater, depending upon the dimensions of the vertical bend.

Points 11 and 12 have been fixed to suit local conditions, as also has the position of the surge tank branch, leaving a distance of 6′-3″ between points 11 and 12 for accommodation of the butterfly valve, which, I hope, will be sufficient. If, however, it is necessary to increase this distance, point 11 should be moved in preference to point 12, and the vertical angle at point 10 adjusted accordingly.

You will note that the survey angle at surge tank between L. P. and H. P. lines has been given as *152 degs.-54 mins.* on *Dg. 90/4/A*, whereas below, on the same drawing, in "Arrangement at Surge Tank," the angle between L. P. and H. P. lines has been given as *152 degs.-23 mins.* The reason for this difference is that the L. P. pipe line and fittings, *i.e.*, butterfly valve, will be set level at the surge tank, whereas there is a slight gradient in the L. P. line up to this point.

No vertical angle has been shown where the change in direction takes place as the change is very small and the pipe joints will be sufficiently flexible to cater for this small angle.

The 18" branch, shown in the centre of the taper pipe at the surge tank, is for the purpose of coupling up to a duplicate pipe, at a later date should the provision of one seem advisable. This branch will be closed by a blank flange until required.

Power-House :—At its lower end the pipe line will terminate with a special flanged taper pipe to fit an existing pipe of 20" bore and I think the flange particulars, given on the drawing, will be quite clear. The 18" branch, shown on the same pipe, is, again, for the purpose of coupling to a duplicate pipe later, and will be closed by a blank flange until then.

The previous Drawings issued, 90/1/A and 90/1/B, are cancelled by the issue of the drawings enclosed.

Certain fittings specified below are not shown on the drawings, as it is to be left to the makers to provide such fittings in the most suitable positions.

Specification

Item 1 : *L. P. Pipe Line.*

Length 1,115′-10″ (including fittings).

Size of pipe 24½″ o/d × ¼″ thick in length or about 16 feet.

Sections 17 to 16 will be laid in a tunnel on pillars and the rest of the L. P. line will be laid below ground, so that two types of joints are to be provided.

Joints :—For *sections 17 to 16*, a length of 350 feet, *victualic* joints with *victualic* closer joint are to be provided.

For *sections 16 to 12* the *Johnson Coupling* joint is to be provided.

Flanged joint, will, of course, be provided, where necessary, for coupling to valves, etc.

Adjustment :—The closer pipe in each section is to be left 4 feet longer than calculated length for cutting to exact length at

site. The type of joint to be supplied will be sufficiently flexible cater for any errors in survey angles.

Bends :—Five bends to be formed on above pipes, of the type specified by the makers and shown on their Drawing No. C. 430, to meet the angles shown on Drawing No. 90/4/B enclosed.

Anchors :—An anchor will be required at *point 16*, but, as the remainder of the L. P. pipe will be below ground, it is considered unnecessary to provide an anchor at each bend.

It is proposed that the vertical bend at *point 11* be anchored, although it has not been shown so on the drawings.

Expansion Joints :—It is proposed that an expansion joint be provided at the upstream side of the anchor at *point 16* and at the downstream side of the anchor at *point 11*, and if the latter introduces any difficulty as regards the arrangement shown on the enclosed drawings, the expansion joint may be placed between the vertical bend and the tapper pipe, the latter, and the 18" sluice valve being moved further downstream

Space at the surge tank site is rather limited, however, and it is desirable that the distance between the butterfly valve and the 18" sluice valve be kept as small as practicable.

Protection :—Sections 17-16 –Pipes to be coated inside and outside, as specified.

Sections 16-12—Pipes to be coated inside and wrapped outside with Hessian cloth drawn through hot solution, as specified by the makers.

Air vent :—It is considered unnecessary to provide an air vent for the L. P. pipe, but, if the makers consider it desirable, you are authorised to provide it, since they have included it in their tender.

If provided, the pipe should reach to a height of *17 feet* above the centre line of the L. P. pipe at the intake, *i.e.*, a level *717 ft.* with respect to the power-house.

Specials :—Such special pipes as are required to meet the conditions and specification are to be provided, whether they have been shown on the drawings or not.

Both bell-mouth pipes are to be provided with puddle flanges.

Item 2 : *24" Sluice valve* :—To be supplied by Messrs. Glenfield and Kennedy in accordance with Messrs. Stewarts and Lloyds' specification and the makers' Drawing No. 18-284/1 and

to be suitable for operation, in the fully closed position, under a head of water of *16'-0"*.

Item 3: *24" Automatic Butterfly valve* :—To be supplied by Messrs. Glenfield and Kennedy in accordance with Messrs. Stewarts and Lloyds' specification and generally of the type shown in the makers' Drawing No. 1131/1933, illustrating a 20" valve.

To be designed to operate in the event of the water velocity in the pipe exceeding 10½ feet per sec. or a discharge of *33 cusecs*

Item 4: *H. P. Pipe line* :—

Total length	...	3,399'-0"	(including fittings)
Size of pipe	...	1,105'-6"	(approx.) 18¾" o/d × ¼" thick in lengths of about 20'-0"
		1,243'-0'	(approx.) 18¾" o/d × 5/16" thick in lengths of about 17'-0"
		1,050'-6"	(aprox.) 18¾" o/d × 3/8" thick in lengths of about 14'-0".

Supports :—The H. P. pipe throughout will be laid above ground on brick or concrete pillars set at every joint, except where the pipes are carried across a ravine, and designed to give support to the pipes at each side of the joints.

Rubbing plates will be required, under item 9, for fitting between the pipes and the tops of the brick supports.

Joints :—Victualic joints, with victualic closer joints, are to be provided throughout, except where special flanges are called for

Bends :—Nine bends, of the type specified by the makers and shown on their Drawing *No. C. 430*, are to be formed on the above pipes to meet the angles shown on Drawings *90/4/A and B* enclosed

Anchors :—Anchor pipes, as specified by the makers and of the type shown on their Drawing No. C. 430, are to be provided at each angle and, in addition, at the following points :—

Midway between points 11 and 10.

Midway between points 9 and 8.

244 feet below point 7.

488 feet below point 7.

It is not convenient to place an anchor midway between points 7 and 6 and, therefore, two will be required.

In their tender, the makers recommended an additional anchor immediately on the downstream side of the ravine crossing,

but the drawings did not show that the upper end of the support for carrying the pipe across the ravine is not more than 10′-0″ below angle 9.

Under the circumstances I suggest that an anchor immediately below the ravine crossing, in addition to the two additional anchors in *sections 9-8*, is really hardly necessary, but, if the makers insist that they consider it advisable, you are authorised to supply it, and in this case it will be situated as follows :—

80 feet below point 9.

There will be, then, 13 anchors, definitely, situated, as stated above, with the possible addition of one more below the ravine crossing.

Similarly it would seem unnecessary to provide an anchor at point 1, since this point is only 20′-0″ from point 2 and 16′-0″ above the branch feeding No. 1 machine in the power-house.

I do not think the makers will propose an anchor at this point but, even if they do, it is not authorised without further reference to me with reasons for proposing it.

Expansion joints :—These will consist of victualic closer joints and they are to be provided on the downstream side of each anchor, closer pipes are to be supplied *4′-0″* longer than the calculated length, as specified by the makers, and will be cut to exact length at site.

Protection :—Pipes to be coated inside and outside, as specified by the makers.

Domestic water supply :—A flanged branch is to be welded to the 18″ main in a convenient position adjacent to the power-house and a suitable stop-cock, loose flange, bolts and jointing material are to be provided to take a heavy water quality 1″ G. I. water pipe for a domestic supply.

Emptying pipe :—A flanged branch is to be welded to the bottom of the 18″ main immediately above the 18″ H. P. sluice valve at the power-house of suitable bore for emptying the pipe line and a suitable stop-cock, short 90 degs. bend, joining material and bolts are to be provided.

Specials :—Such special pipes as are required to meet the conditions and specification are to be provided, whether they are shown on the enclosed drawings or not.

Both 18″ branches are to be fitted with blank flanges.

Item 5 : *18″ Sluice valve at surge tank* :—To be supplied by Messrs. Glenfield and Kennedy in accordance with Messrs. Stewarts and Lloyds' specification. To be suitable for operation, in the fully closed position, with pressure amounting to a head of water of about 33 feet on one side only.

Item 6 : *Air vent* :—One $6\frac{1}{2}''$ o/d × $\frac{1}{4}''$ thick air vent pipe to be provided immediately below the 18″ sluice valve at surge tank, suitable flanged branch to be welded to the 18″ main at a suitable angle to bring the vent pipe vertical.

The top or the vent pipe to be not lower than 717′-0″ above power-house level.

Item 7 : *Support for ravine crossing* :—This is to be designed, in the first instance, for carrying a duplicate pipe which may be provided later and the design shown on the makers, Drawing No. N. 569 is preferred to the bridge shown on their Drawing No. N. 570 and, if there is no difficulty in adapting this design to the purpose of carrying a second pipe line of the same size, it is to be supplied with suitable modifications to meet the revised conditions.

If, however, the makers prefer, for technical reasons, the bridge shown on Drawing No. N. 570, you are authorised to supply it at the small extra price asked.

Item 8 : *18″ H. P. Sluice valve at power-house* :—To be supplied by Messrs. Glenfield and Kennedy in accordance with Messrs. Stewarts and Lloyds' specification and the makers' Drawing No. 1130/1933.

To be provided with suitable bye-pass pipe and stop cock, which may be fitted to the valve body itself or to the pipes on either side of it.

The valve to be suitable, in any case, for operation in the fully-closed position, under a head of water of 716 feet, as specified.

Item 9 : *Rubbing Plates* :—Rubbing plates of suitable design and in sufficient quantity for placing between the pipes and supporting pillars are to be provided.

Plates are preferred to rollers.

It is hoped that the information contained in the enclosed drawings and the above specification is complete, and will enable you to have the work put in hand immediately and allow it to continue uninterruptedly, but, if any points do arise about which there is any doubt, please let me know immediately.

Makers' drawing :—You are requested to supply, as soon as possible, certified dimensioned drawings of all fittings and special pipes and drawings showing arrangements, with dimensions, at surge tank and power-house.

Detailed drawings and full instructions will be required later for erection of the automatic butterfly valve.

Delivery :—There has been a good deal of delay on the part of makers in submitting their tenders and the matter is now exceedingly urgent and I shall be glad if you will do all you can to hasten delivery.

Weights :—It is noted that some of the individual weights will exceed to cwt. and transport arrangements will be made accordingly.

Packing :—Please have valves and other fragile parts very carefully packed and corrodible parts adequately protected.

Marks : –Individual parts of valves and other fittings are to be carefully marked for identification purposes and to facilitate erection.

Packages are to be marked as follows :—

Advice notes :—Detailed advice notes, giving particulars of the marks of individual parts, are required in good time in triplicate.

Consignment :—Goods to be consigned to this Department c/o. Messrs. Gladstone Wyllie & Co., as usual.

Insurance :—Goods to be insured against loss and damage however caused to destination.

Payment :—Payment will be made against your draft as usual.

CHAPTER XI

TURBINES

Turbines, their classification and conditions of operation :— Hydraulic prime movers may be classified into various ways. They have, in common, a wheel to which buckets are attached. The wheel is arranged to revolve in a fixed case having attached to it a nozzle, guide or series of guides. The guide passages or the nozzles direct the water at a suitable angle into the buckets of wheel. The revolving wheel or passages, whose functions are to receive the water, utilise its energy and discharge or wash it as nearly devoid of energy as possible.

The classification may be on different basis :—

(I) With reference to the action of the water on the same reaction or pressure turbines : (1) this is completely filled with water or drowned and the water acts, by its own pressure, *e.g.*, Francis' turbine.

Conditions of operation—**guides, complete** :—

(*a*) Buckets with restricted outlets.

(*b*) Buckets or wheel passages completely filled.

(*c*) Energy most largely developed through reactive pressure.

(*d*) Discharge usually below tail water or into a draft tube.

(2) Impulse or velocity turbines. These are put into action by the head which is connected in velocity, *e.g.*, Pelton, Gerard.

Conditions of operation—**guides, partial or complete** :—

(*a*) Buckets with outlet free and unrestricted.

(*b*) Wheel passages never filled.

(*c*) Energy entirely due to velocity.

(*d*) Discharge must be above tail water.

(*e*) No draft tube possible except with special arrangement which will prevent contact of tail water with wheels.

(3) *Limit turbines*, which may act either by reaction or impulse. Limit turbines form the dividing line between reaction and impulse wheels and these combine many advantages of both the types. They may be considered as impulse turbines with free deviation.

Conditions of operation—

(*a*) The buckets are so designed that the discharge is unrestricted where above tail water.

(*b*) Buckets in this case, just filled, act primarily by effect of velocity.

(*c*) Discharge above tail water.

(*d*) If tail water rises to buckets, the discharge is restricted and pressure reaction results. In this case the full bucket admits reaction and discharge may be below tail water.

(II) According to the direction of flow in reference to the wheel.

(*a*) *Radial flow turbine* :—In these, the water flowing through the wheel is in a radial direction.

(*i*) Outward radial flow turbines.

(*ii*) Inward radial flow turbines in which the water flows in a radial direction such as Francis.

(*b*) *Parallel or axial flow* :—The direction of flow being parallel to the turbine axis such as Gerard.

(*c*) *Mixed flow turbines* in which the flow is partially radial and partially axial.

(III) In reference to the *position of the wheel shaft* :—

(*a*) Vertical ; (*b*) Horizontal.

(IV) In accordance with the *arrangement of nozzles* or guides :—

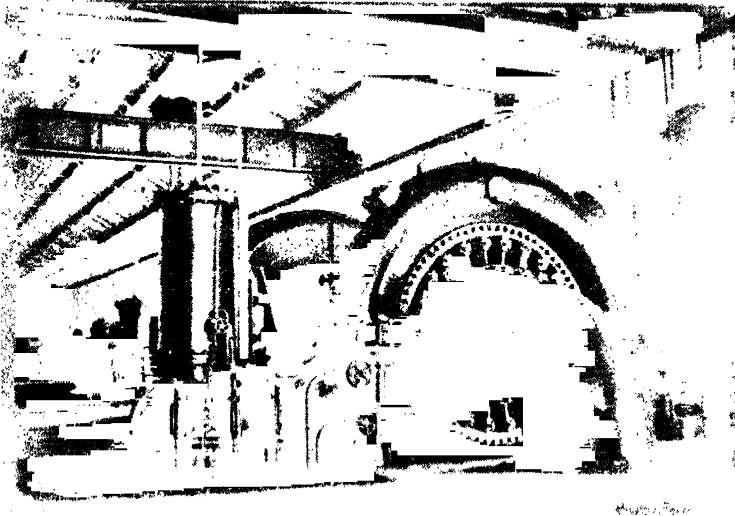
(*a*) Complete turbines with guides surrounding the entire wheel.

(*b*) Partial turbines with guides partially surrounding the wheel in one or more groups.

The reaction turbine is a combined potential and kinetic energy wheel. It admits water all around the periphery of the runner and all parts of the same perform useful work. This type of turbine consists essentially of a wheel or runner provided with vanes into which the water is directed by a series of guide vanes. The water, on passing these guide vanes, is under pressure and supplies energy partly in the kinetic and partly in the pressure form. In its passage through the runner the pressure energy is utilised in increasing the relative velocity of flow between the vanes, and the water finally leaves the runner at the pressure obtained in the discharge pipe or draught tube.

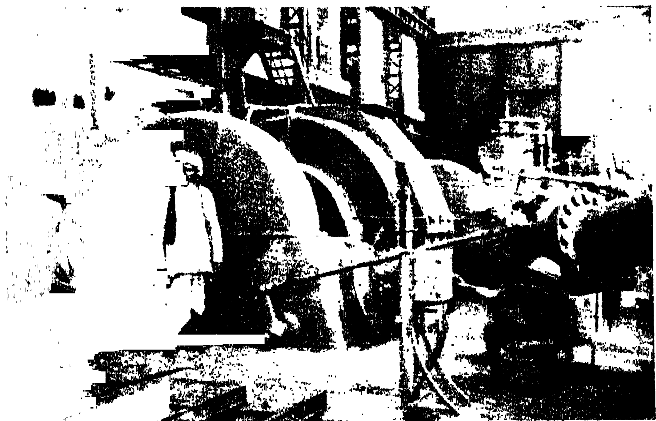
The water may pass radially inwards or outwards, or it may enter the runner radially towards the shaft but leaves in an axial direction, *i.e.*, in a direction parallel with the shaft. In this case, the turbine is of the '*mixed flow*' type.

Fig. 1.



6,000 kW. Alternator and the 9,000 B. H. P. Boving Reaction Turbines.

Fig.

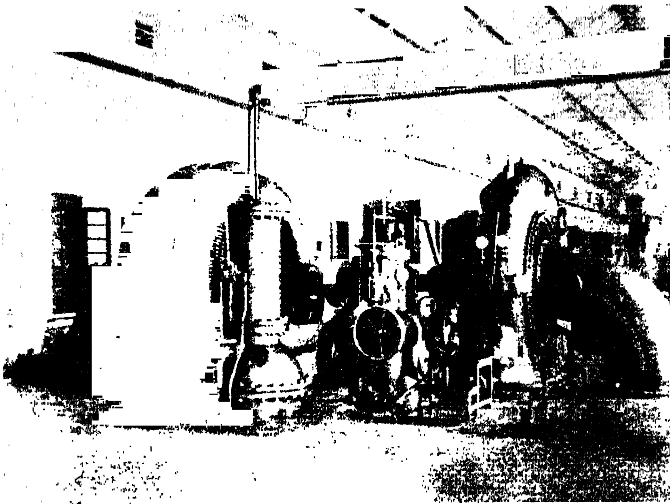


View showing the complete unit comprising the Turbine, the pipe unit, governor and generator, Pykara

The runner rotates partly from velocity action and partly from reaction due to pressure and consequent acceleration in the buckets. As the draught tube is closed, the runner is full of water and practically the total difference in head between head water and tail water is useful.

In an impulse turbine the whole head of the supply water is converted into kinetic energy before the wheel is reached. The water, discharged from one or more nozzles, impinges on a number of buckets attached to the periphery of the runner

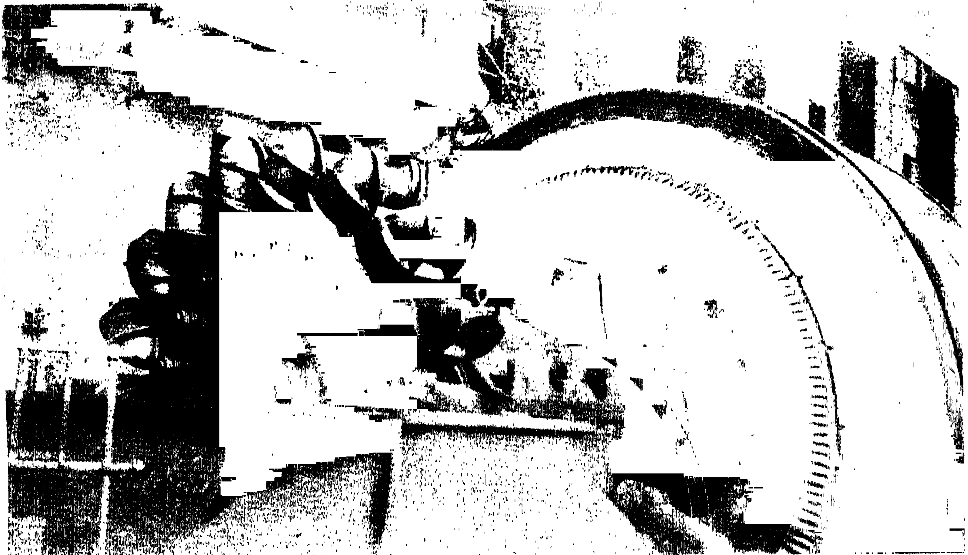
Fig. 3.



The 3,000 kW. alternator and 5,600 B. H. P Boving Reaction Turbine and governor and draft tube.

and the momentum of the mass of water in its impulse upon the runner bucket is, therefore, the main principle utilised in the energy transformation. Its direction is freely deviated by the buckets, and its pressure remains uniform during its passage through the turbine. When the water leaves the buckets, it has so little absolute velocity that practically its entire energy has been imparted to the runner. The water, leaving the nozzles, is under atmospheric pressure, the pressure inside the turbine casing being usually atmospheric.

Fig. 4.



End view of impulse Turbine opened for overhaul

Since, usually, the number of nozzle is small, as compared with the number of buckets, the latter are in active use only during a part of each revolution and hence this type of prime mover is sometimes called WATER-WHEELS instead of TURBINES.

The *peripheral velocity of a Pelton* wheel for maximum efficiency is slightly less than half the spouting velocity of the jet usually approximately $\cdot 46\sqrt{2gH}$, (where H = the head); while that of the *reaction turbine* varies from about $0{\cdot}65\sqrt{2gH}$ to $1{\cdot}45\sqrt{2gH}$ depending on the particular design of it. Because of this the Pelton wheel is well adapted for very high heads, which may then be utilised with moderate speed of rotation. On the other hand, the relatively high speed of the reaction turbines enables reasonably high relative speeds to be obtained with low heads.

The Pelton wheel cannot well be designed to utilise efficiently more than two jets on a single wheel, and as the maximum practicable jet diameter is about 12 inches or so, the volume of water which can be handled and the output of the turbine becomes small under low heads. The reaction turbine with its full peripheral admission is well adapted for large volumes. It is not suited for small powers under high heads since the volume of water is small, the water-ways are of very small sectional area and easily become choked by floating debris and the fluid friction losses become relatively high.

The Pelton wheel is not well adapted to be used with a suction or draught tube, and where the tail-race level varies appreciably, it must be installed above the highest tail water level with some sacrifice of head. The reaction turbine lends itself readily to this construction and has the further advantage that it may be drowned without loss of efficiency. The efficiency of the reaction turbine is not so sensitive to changes of head as the Pelton wheel is, and since the percentage variation in the head is usually greater in low heads than in the high-head plants, this is another reason why the Pelton wheel is not well adapted for low heads.

If operated under constant head and constant speed, the efficiency of the Pelton wheel does not fall off so rapidly at part loads as that of the reaction turbine. It has a slightly higher full-load efficiency, so that the average efficiency from half to full-load is sensibly the same in a well-designed machine of either type.

In general, the full-load efficiency does not greatly exceed 85% in the case of a large reaction turbine and 80% in the case

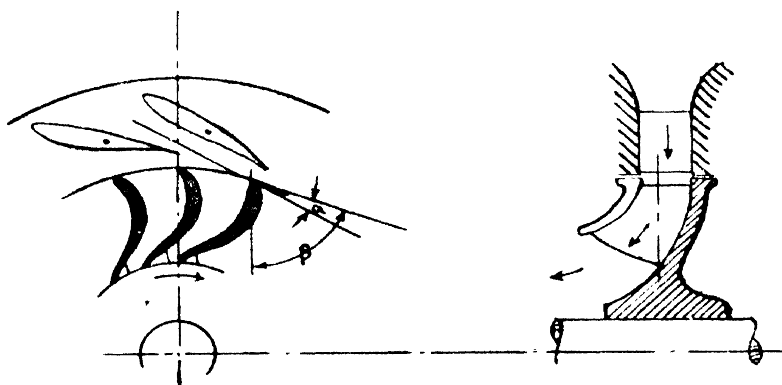


Fig. (5)

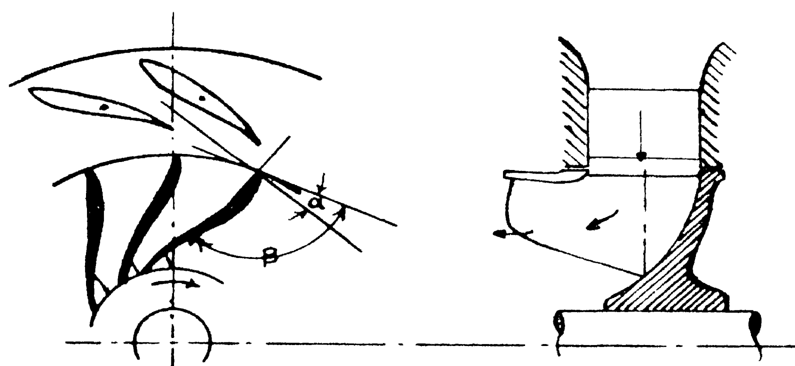


Fig. ([illegible])

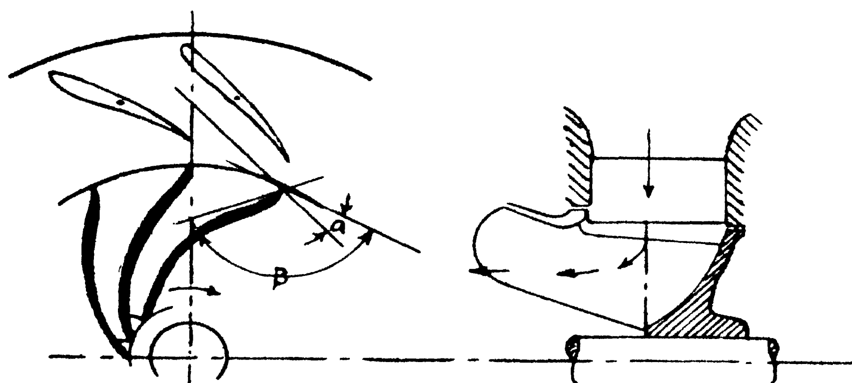


Fig. ([illegible])

of a large Pelton wheel. The possibilities of accurate speed regulation are about equal in the two types.

For large units, the reaction turbines are generally preferable for heads up to 450 ft. For heads above 750 ft. the Pelton wheel is more suitable, while between these limits the choice depends largely upon local circumstances and on the power required.

Variation of speed of reaction turbine can be effected by variation of (1) the runner diameter, (2) the bucket angle, (3) the angle between the entrance speed and the peripheral speed.

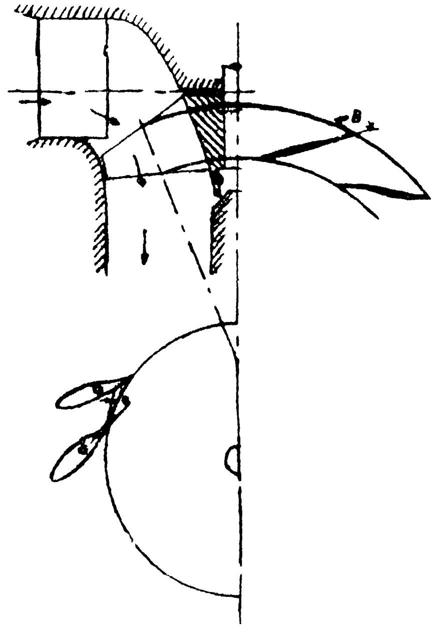


Fig. 5.—Set of figures in page 446 shows low-speed runner for Francis' reaction turbine used for relatively high heads and small quantities of water. The bucket angle is nearly 90° and the angle α of the water leaving the guides is small.

Fig. 6 represents medium-speed runner, the angle β is approximately 90° and the angle α of the water leaving the guides is small.

Fig. 7 shows a high-speed runner for low heads and relatively large quantities of water. The angle β is greater than 90° and the angle α is also greater than before, thus giving a very high peripheral velocity.

Fig (8)

Fig. 8 represents a very high-speed runner of the propeller type. The angle β being much larger than 90° and the angle α is also considerably larger than before.

Hydraulics of the Turbines :—(1) In impulse wheel

$$\phi = \frac{v'}{v} = \frac{vL}{\sqrt{2gh}} \text{ and } v' = \phi v \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

where, v' = velocity of the periphery of the wheel.

v = velocity of the jet or spouting velocity of the water under an effective head.

ϕ = peripheral coefficient.

In determining the force F, exerted upon the moving bucket, the relative velocity, instead of the actual velocity of the jet, must be considered.

(2) The relative velocity $V_r = v - \phi v = (1 - \phi) v$ ... (2)

The relative weight of water that strikes a single bucket per second will be less on account of the movement of the buckets, but as new buckets constantly intercept the path of the jet, the total amount of water effective is equal to the total discharge of the jet.

But if F = the force producing the pressure or motion.
Q = discharge in cu. sec.
v = theoretical spouting velocity of water in feet per sec. = $\sqrt{2gH}$
g = acceleration of gravity.
α = angle of deflection of jet.
W = work done upon the buckets.
w = weight of unit cu. ft. of water = 62·5 lbs.

(3) $F = \frac{Q\,wv}{g} - \frac{Q\,wv}{g} \cos \alpha$

$= \frac{Q\,wv}{g}(1 - \cos \alpha)$ ... ... (3)

From equations (2) and (3)

we get $F = (1 - \cos \alpha)\frac{Q\,vw}{g}(1 - \phi)$ ... ... (4)

(4) The work W done upon the bucket per second is equal to the force F times the distance ϕv through which it acts and is

$W = F\phi v = (1 - \cos \alpha)(1 - \phi)\frac{Qwv}{g}\phi v$ ... ... (5)

This is maximum when $(1 - \phi)\phi$ is a maximum.

Differentiating and equating to zero

$2\phi - 1 = 0$

or, $\phi = \cdot 5$

Substituting $\phi = \cdot 5$ and $\alpha = 180^\circ$ in equation ... (5)

$W = \frac{Qwv^2}{2g}$

W, the energy, equals to the entire energy of the jet; hence, the theoretical efficiency, when $\phi = \cdot 5$, is cent per cent.

Another criterion for maximum efficiency is that the absolute velocity of water leaving the bucket must be zero.

When $\alpha = 180$, the absolute velocity with which the water leaves the bucket is evidently the velocity relative to the bucket *minus* the velocity of the bucket,

or, the absolute velocity $= (1-\phi)\,v - \phi v = v - 2\phi v = 0$

or, $\phi = 0{\cdot}5$.

Relation of speed to diameter :—

v′ may also be expressed in terms of the diameter of the wheel d and the number of revolutions R.

$$v' = \frac{d\,\pi\,R}{12 \times 60} = \frac{3{\cdot}1416\,dR}{720} \quad \ldots \quad (6)$$

$$\therefore \phi = \frac{3{\cdot}1416\,dR}{720 \times 8{\cdot}025\,\sqrt{h}} = {\cdot}0005437\,\frac{d\,R}{\sqrt{h}} \quad \ldots \quad (7)$$

$$N = \frac{\phi\,\sqrt{h}}{{\cdot}0005437\,d} = \frac{1840\,\phi\,\sqrt{h}}{d}$$

The speed coefficient may be deduced as follows :—

$$\frac{dR}{\sqrt{h}} = 1{,}840\,\phi = \triangle, \text{ the speed coefficient, } i.e.,$$

the speed or R. P. M. of one inch-wheel under one-foot head.

If the head is one foot, or $h = 1$

$dN = 1{,}840\,\phi = \triangle$

If $d = 1$ inch, the equation becomes

$N_1 = 1{,}840\,\phi =$ revolutions per minute of an one-wheel under one-foot head.

Effect of angle of discharge on efficiency :—

In impulse turbine the direction of water cannot be practically changed through 180° as it would interfere with succeeding bucket. Thus α must be less than 180° and the absolute velocity of water in leaving the bucket cannot be zero. The loss from this source is very small as α may differ considerably from 180° without much effect on the bucket pressure and hence on the efficiency.

The ratio of actual pressure, when α is less than 180°, to maximum possible pressure, when $\alpha = 180°$, is

$$\frac{(1-\cos\alpha)\ QW\ (1-\phi)\ \frac{(1-\phi)\,V'}{g}}{(1-\cos 180°)\ QW\ (1-\phi)\ \frac{(1-\phi)\,V'}{g}} = \frac{1-\cos\alpha}{2} = \frac{1+\cos\beta}{2}$$

If $\beta = 8°$, $\alpha = 172°$ and $\frac{1 - \cos \alpha}{2} = \cdot 995 = 180$ Z

where, β = supplement to angle of deflection.

Showing only 0·5 per cent. reduction, the efficiency is also affected in the same ratio.

Example :—If a series of turbines are designated to operate with a speed coefficient of 849. Find ϕ and number of revolutions per minute of an 80″-wheel of this series under one-foot head and also under 1,668, feet speed.

$$\phi = \frac{840}{1,840} = 4 \cdot 5$$

(2) The number of revolutions per minute of a 40-inch wheel of this series under one-foot head will be—

$$N = \frac{840}{80} = 10 \cdot 5$$

(3) The number of revolutions of the same wheel under 1,668 feet will be—

$$N = N_1 \sqrt{h} = 4,028 \cdot 6$$

Discharge diameter of the runner is of great importance on the entire plant layout—

Let d_1 = inlet diameter of the runner.

d_2 = outlet diameter.

and $\delta = \frac{d_2}{d_1}$

$$N = \frac{60 v'}{\pi d_1} = \frac{60 \delta \phi \sqrt{2gH}}{\pi d_2}$$

But, $N = \frac{N_s H^{\frac{5}{4}}}{\sqrt{P}}$

$$\therefore \frac{N_s H^{\frac{5}{4}}}{\sqrt{P}} = \frac{60 \delta \phi \sqrt{2gH}}{\pi d_2}$$

$$\therefore D_2 = \frac{153 \cdot 2 \delta \phi \sqrt{P}}{N_s H^{\frac{5}{4}}} \text{ feet.}$$

The value is very nearly a straight line function of N_s and is very nearly equal to

$$\delta = \frac{N_s}{100} + 0 \cdot 55, \text{ approximately.}$$

Runaway Speed:—The governor of a turbine maintains constant speed of a turbine under operating conditions. But, if the load changes without a corresponding change of the admitted quantity of water, the speed varies and becomes greater as the load decreases. If the load suddenly drops off and the gates remain wide open, the speed rises very high and results in disaster to the direct-connected generators, which must be designed to withstand such runaway speed of the water wheels.

For high-head plants with impulse wheels the runaway speed should be preferably double the normal speed. For low heads, with Francis' reaction turbines, at the most efficient speed and a constant head, the runaway speed may be 55 to 85 per cent. above the normal speed.

Under low-head conditions with a wide variation in the head and with the wheels designed for an intermediate speed to work under these different conditions, the runaway speed up to three times the normal may be realised under the maximum head. This depends essentially on the range of head and on the design of the turbine. If the specific speed is well-selected, there will be no pitting for any set of conditions for which a runner may be designed.

Runaway speed of wheels becomes of importance when considering the design of rotary machinery to be connected thereto. If such machinery is built only of sufficient strength to stand the regular speed for which the unit is designed to run, it may be greatly over-stressed and, perhaps, destroyed, if through any mischance the governor should be broken or disabled and the gates of the wheel should open and the turbine acquires its runaway speed. They should thus be built for the highest speed under which they may be called upon to operate. As the friction intervenes and v' at its maximum can seldom have a velocity at runaway or no-load speed or more than 84 to 92 % of v, the actual value of v' for the greatest efficiency of the tangential wheel will be from 42 per cent. to 46 per cent. of v, that is,

$$\phi = v'/v = \cdot 42 \text{ to } \cdot 46.$$

In reaction turbines the maximum or runaway speed will vary from 110 to 160 % of v and the value of v' for greatest efficiency will be from 60 to 95 %.

Troubles

Dangerous runaway speed.—*Remedy*:—(1) A fly ball mechanism independent of the turbine governor is driven from

the shaft of the unit, which, in the event of the excessive speed by means of control valves, admits water behind the piston in an auxiliary cylinder on the governor. This causes it to move in such a manner as to overcome the oil-pressure in the control element of the governor and shut down the unit. (2) The best procedure is to so design the governor and generator so that no such excessive stresses may be developed which they cannot stand.

In the Shannon power-house (Mundis) the highest possible runaway speed of the turbines that can be obtained under the maximum head has been taken to be 860 R. P. M., the normal speed being 428·5 R. P. M.

Pitting :—Pitting of runners takes place while working and the metal forming the buckets or blades disappear.

Cause:—Chemical or electro-chemical. Pitting occurs more rapidly when there are high velocities and high vacuum. The height above tail water or the vacuum on the runner seems to influence pitting greatly. The design of the runner may have a great effect on pitting.

Remedy: —Pitting runners can be repaired by welding in additional metal when the runner material is of cast iron or cast steel.

Pitting is sometimes stopped by drilling a small hole through the runner blade at the pitted section.

Specific Speed of a Turbine:—The specific speed of a turbine is the speed at which it will run when producing one horse-power under a head of one foot of water. This is sometimes called UNIT SPEED or TYPE CHARACTERISTIC of the turbine. Knowing specific speed, type of turbine can be judged, as each type has its own value between certain limits. Assuming the turbines to be similar, their linear dimensions in proportion and blade angles constant and applying the principles of similarity, we get the following equation :

Let D = diameter of turbine in feet.
n_s = specific speed in R P. M.
P = horse-power developed.

Then, $v = \frac{wD}{2}$ from which $D \propto \frac{v}{w}$

where, w = angular velocity in radians per sec.
v = tangential velocity in ft. per sec.

But $w \propto n$ = the number of revolutions made by the wheel per minute.

and $v \propto V$ or $v \propto \sqrt{H}$, since $(V \propto \sqrt{H})$

where, H = head in ft.
V = velocity entering water in ft. per sec.

Hence, $D \propto \dfrac{\sqrt{H}}{n}$ ... ... (1)

Also $v \propto D$ from assumption.

Hence, $v \propto \dfrac{\sqrt{H}}{n}$ from (1) ... ... (2)

where, v = breadth of runner in ft.

Now velocity of flow $V_f \propto V$

i.e., $V_f \propto \sqrt{H}$, as $V \propto \sqrt{H}$ ... ... (3)

The quantity of water passing through turbine = radial area of flow × velocity of flow $= \pi D b \times V_f$

Hence, from (1), (2) and (3)

$$\text{quantity per sec} \propto \frac{\sqrt{H}}{n} \times \frac{\sqrt{H}}{n} \times \sqrt{H}$$

$$\text{or,} \propto \frac{H^{\frac{3}{2}}}{n^2}$$

Weight of water passing per sec. = W = 62·4 × quantity per sec.

or $W \propto \dfrac{H^{\frac{3}{2}}}{n^2}$.

Now horse-power of turbine $= \dfrac{WH}{550}$

Hence, $P \propto \dfrac{H^{\frac{3}{2}}}{n^2} \times H$

$\propto \dfrac{H^{\frac{5}{2}}}{n^2}$

or, $n \propto \dfrac{H^{\frac{5}{4}}}{\sqrt{P}}$, *i.e.*, $n = k \dfrac{H^{\frac{5}{4}}}{\sqrt{P}}$, where k is a constant depending on the type of turbine.

When turbine works under a head of 1 ft. developing one H. P.

K equals n_s under the conditions, so that

$$K = n_s = \frac{n\sqrt{P}}{H^{\frac{5}{4}}}$$

The specific speed N_s of Pelton wheel lies between 3 and 20 and that of Francis' type between 20 and 120; these may have N_s as low as 20 to 30 on heads of 400 feet or more up to several hundreds on very low heads.

Example.—At Shannon power-house, the turbine has a capacity of 17,000 B. H. P., the height is 1,668 ft. and the normal speed is 428.5 R. P. M. Find N_s.

Solution :—

$$N_s = \frac{N\sqrt{P}}{H^{\frac{5}{4}}} = \frac{428\cdot5\sqrt{17000}}{1\cdot668^{5/4}} = 5.25$$

This is within the range.

If N_s were more than the limit, say, 20, the greater number of units should have been selected for the same power and the same type of turbine. Note that runners of low specific speeds are suitable for high heads and those of high specific speeds are suitable for low heads. The combination of head and specific speeds is such as to keep the greater speeds within reasonable range for all values of heads.

THE SPECIFIC SPEED OF A TURBINE WITH MORE THAN ONE RUNNER OR NOZZLE.—The horse-power in the above formula is the output from each runner. In the case of a turbine of the same capacity, having n runners of the same specific speed, the R. P. M. would be $\sqrt{n}$ times the R. P. M. of a single runner turbines further for a given value of R. P. M. and H, the output in H. P., is the square of the specific speed and for a given head and H. P., the R. P. M. of a turbine is proportional to the specific speed.

General formula from which the maximum safe specific speed for any given head may be computed.

$$N'_s = \frac{5050}{H + 32} + 19$$

where, N'_s = the maximum safe specific speed.
H = head in feet at which unit will operate.

If the necessary output and speed are known, we get the characteristic speed—

$$N_s = \frac{N\sqrt{P}}{H^{\frac{5}{4}}}$$

where, N_s = specific speed.
P = horse-power per runner.
H = head in feet at which unit is operated.

Selection of Turbines :—This is with regard to

(1) Number of turbines.

Number—For single-plant systems the number of units should preferably be not less than three or four. If more are necessary, the number should be governed by the upper limit in design considered both from technical and economical standpoint.

Have units of the same size, if possible, and have reserve sets for back water and other emergency conditions.

(2) Capacity of the unit.

(3) Speed of the unit.

In selecting the speed of a hydro-electric unit the cost and efficiency of the generator must be taken into consideration. For the average size generator the cost decreases as the speed increases to a certain point.

(4) Type of turbine.

Here the selection will depend upon :—

(1) The combination of the turbine and generator set.
(2) The head and its variations.
(3) Stream flow.
(4) Storage facilities.
(5) Limitation of turbine design.
(6) Load factor.
(7) Nature of the load.

Where the load variations are large and where the quantity of water is limited, the turbine should be so selected that it will give its maximum efficiency under several conditions of operation. It is usually necessary to select a turbine having a given speed and output in order to operate a generator for which these characteristics are fixed. Here the turbine should be designed to give as nearly as possible the exact output required to work at approximately at full load and hence under the most economical conditions. It is advisable to split up the installation into as few units as the operating conditions permit.

(8) Reserve capacity.
(9) The reliability and flexibility of service.
(10) Cost and operating expenses.
(11) Ultimate development.

Select the units—its number and capacity—as near the full load as possible. Increase the number as the load increases.

The type of turbine should not have N_s, a value less than the desired speed, and for high efficiency not greatly different from

this value. If the calculated value for a single turbine is greater than is attainable with the type selected, the power must be divided between two or more units until the required conditions are satisfied.

Homologous Equations :—If the efficiency, speed, power and discharge diameter of a runner at a given head is known, the following equations will give directly the speed, power, discharge of homologous turbines under different conditions, *i.e.*, two having the same N_s efficiency, or single turbine of operated under different conditions.

For constant head :—

1. If N, Q, d, e are constant, then $\frac{P_2}{P_1} = \left(\frac{H_2}{H_1}\right)^{\frac{3}{2}}$

2. If P, Q, d, e are constant, $\frac{N_2}{N_1} = \left(\frac{H_2}{H_1}\right)^{\frac{1}{2}}$

3. If N, P, d, e are constant, $\frac{Q_2}{Q_1} = \left(\frac{H_2}{H_1}\right)^{\frac{1}{2}}$

4. If H, N, q, e are constant, then $\frac{P_2}{P_1} = \left(\frac{d_2}{d_1}\right)^{2}$

5. If H, P, Q, e are constant, $\frac{N_2}{N_1} = \frac{d_1}{d_2}$

6. If H, R, P, e are constant, $\frac{Q_2}{Q_1} = \left(\frac{d_2}{d_1}\right)^{2}$

7. If H, N, p, e are constant, then $\frac{p_1}{p_2} = \frac{Q_1}{Q_2}$

where, p_1 and p_2 = horse-power for different conditions.

d_1 and d_2 = runner diameter in inches for different conditions.

H_1 and H_2 = head in feet for different conditions.

Q_1 and Q_2 = discharge in cu. secs. for different conditions.

R_1 and R_2 = revolutions per minute.

e = efficiency.

Example :—The turbine at Shannon generating station works under an effective head of 16,668 feet and gives 17,000 B.H.P. when

running at the normal speed of 428·5 R. P. M. efficiency 87 %. Find the other operating characteristics when the effective head will be raised to 2,001 feet.

$$\text{On 1,668 ft. } Q = \frac{17{,}000 \times 550}{\cdot 87 \times 62{\cdot}5 \times 1{,}668} = 103 \text{ cu. sec.}$$

$$\text{On 2,001 ft. } P = 17{,}000 \sqrt{\left(\frac{2{,}001}{1{,}668}\right)^3} = 22{,}340 \text{ h. p.}$$

$$Q = 103 \sqrt{\left(\frac{2{,}101}{1{,}668}\right)} = 112{\cdot}8 \text{ cu. sec.}$$

$$N = 423{\cdot}5 \sqrt{\left(\frac{2{,}001}{1{,}668}\right)} = 469{\cdot}1 \text{ R. P M.}$$

The characteristics of different wheels can best be judged by a critical comparison of their *specific speeds* without considering their actual speed, power or head. As deduced before, the specific speed of any turbine is given as—

$N_s = \text{R.P.M.} \times \frac{\sqrt{\text{H P.}}}{h^{\frac{5}{4}}}$, where h is the effective 'fall'. Here the one factor affecting N_s is R.P.M—h, of course, being constant —and it has its own limitations set up by practical considerations.

With low-head developments the highest speed compatible with good practice is to be selected to keep down the weight and consequently the cost of generator. With very high heads, as, of course, in Pykara, it is mostly a question of speed being kept reasonably slow so as to avoid the use of costly high-speed generators. It is the designer and the progress of his art that set up the limitations. The limit of high speed for low heads is limited by the progress of the art of designing high-speed runner ; and the limit of low speed under high heads is fixed by the risk involved in designing runners for operation with very low coefficient of specific speed.

For the auxiliary or Glen Morgan Scheme a speed of 750 R.P.M. is used, the head being 765 ft. But, for the main scheme, a speed of 500 R.P.M. is specified, the head being 3,090 ft.; the main reason for choosing 500 R.P.M. being the higher efficiency possible in this case—some 2 % extra over machines designed for this head and 750 R.P.M. The question of the increased cost of the unit and the transportation of the heavier piece, which this choice of speed necessitates, may, perhaps, offset, to some extent, the gain in efficiency; but questions of regulation

and lack of proper guarantee and the conservative tendency to take no risks would also justify and advocate the choice of 500 R.P.M. The specific speed in the two cases are :—

$$\text{Glen Morgan } N_s = 750 \times \frac{\sqrt{500}}{765^{\frac{5}{4}}} = 4 \text{ (approx.)}.$$

$$\text{Pykara } N_s = 500 \times \frac{\sqrt{10{,}000}}{3{,}090^{\frac{5}{4}}} = 2 \text{ (approx.)}.$$

(Impulse wheels work efficiently under specific speeds of 4 to 1, maximum specific speed at loss of efficiency $6\frac{1}{2}$ to $7\frac{1}{2}$. Reaction turbines work above 20 specific speed and work efficiently between 25 and 75). Thus, in both the cases, the choice of impulse wheels is justified.

The maximum full-load capacity of a turbine is that point beyond which the output decreases with an increase in gate opening. The margin between maximum efficiency and maximum capacity, as can be seen from characteristic curves, depends upon the specific speed of the runner, and is smaller the higher the specific speed. The specific speed may thus be increased to such an extent that the point of maximum efficiency and maximum output coincide. There seems to be some flaw in the common reasoning of choosing maximum output. The units, at least so of Pykara and must be so of many stations, will be operating at 80 % full load or near about for most of the time than 100%, and so it would appear economic to fix the specific speed such that the maximum efficiency occurs near about 80% full-load output. These considerations would, to a great extent, adjust the values of R.P.M. and output of the units to be fixed. This is the realm of the designer and the manufacturer.

Impulse turbines of the *horizontal* type are used in Glen Morgan and specified for Pykara. The relative advantages of the 'horizontal' and the 'vertical' type seem to outweigh each other. The horizontal design is claimed to be most economical with advantages of simplicity of constructions and arrangements of parts available for inspection, lubrication and cleaning. The vertical unit, in its turn, is claimed to be much neater unit of slightly lesser losses, lending itself for better ventilation and occupying less space in the power-house, and particularly advantageous when the water contains large quantities of dirt, etc.; with the vertical type, also up to 6 jets can be installed in a single wheel. Besides all these, practical considerations of speed,

design and satisfactory operation must have caused the choice of horizontal units. The vertical ones seem to be used only for low-head developments.

Buckets : In the Pelton wheels, used in the Glen Morgan Scheme, the wheel centre consists of a single rim and the buckets have two ‘ lugs ’ or handles which are machined to a press fit over the rim of the wheel centre and held in position by two bolts with nuts on both the sides.

The buckets are ellipsoidal, which cause the water jet to impinge without shock or disturbance, and it is discharged along natural lines over the entire bucket surface. The central portion of the front entering wedge or lip of the bucket is cut away in the form of a semicircular notch, and this opening allows the solid circular water jet to discharge upon the central dividing wedge of the bucket, without being split in a horizontal plane, with the result, that all eddy currents are avoided and the full force of the jet is expended for useful working resulting in the maximum bucket efficiency. That there is the reason for the semicircular cut of the buckets. A jet strikes three buckets.

In the other kind of buckets there will be three ‘lugs,’ the wheel rim being of the U-shape. The central lug of one bucket and the outer lugs of the next bucket are in a line. The central lug is a close fit between the two rims and the two rear lugs shaddle the rims and the three are bolted together by a pair of bolts.

Pelton Wheels at Mahora

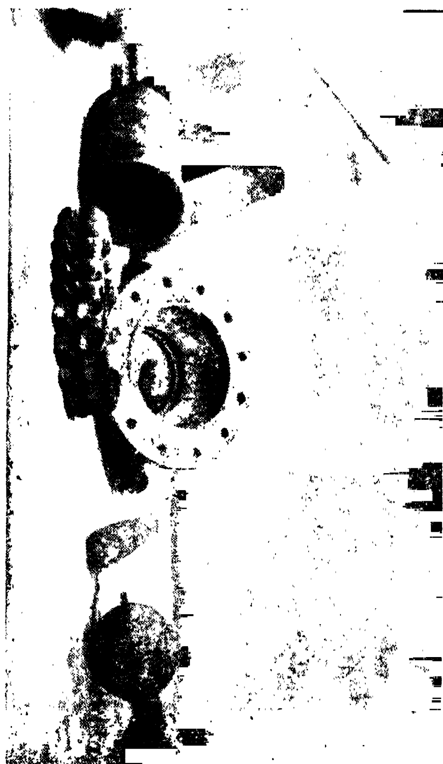
This type of wheel has been evolved from an earlier type of water wheel used in the mines of California. It works by the velocity of the jets striking the buckets, and hence it is only used on high heads.

The jet impinges on the wheel from two nozzles (sometimes one only) and strikes the blade at the centre flowing axially in both direction. The line of action of each jet is tangential to the pitch circle of the buckets, hence the machine is also termed as a tangential impulse wheel.

The buckets consist of double hemispherical cup. As the water flows axially in both direction, there is no axial thrust on the wheel.

In order that the discharge from one shall clear the back of the following bucket, the jet is deflected 160°.

Fig. 9.



Parts of Pelton wheel, Mahora.

As the Pelton wheel works by the velocity and not by pressure of water, so the casing of the wheel contains air at the atmospheric pressure and water, after striking the buckets, runs off freely by gravity and not in a pipe or draft tube. The height of the tail race does not enter into the calculations of the head.

The two jets are placed at an angle of 60° apart.

Owing to the splash from one jet affecting the other and to the fact that one of the nozzles must be at a greater elevation than the lowest point of the bucket pitch circle, the efficiency is somewhat lower than the single jet machine, and where the power required from a unit is greater than can be obtained from a single jet, it is usually preferable to mount two single jet wheels side by side on the same shaft.

Nozzles: The modern Pelton wheels are always fitted with a circular nozzle having a needle regulator. This is a cylindrical needle or spear of tapering section fitted inside the nozzle axially with the jet. The discharge takes place through the annular space between the needle and the nozzle, giving a solid cylindrical jet on leaving the needle.

The size of the jet is determined by the axial position of the needle. This may be governed by hand or by governing mechanism. In Mahora power-house this is hand-regulated.

It is most necessary that the needle should be central in the nozzle and should be rigidly supported so as to prevent all vibration.

The governor operates the deflectors which do not effect the size of the jet but only change its direction.

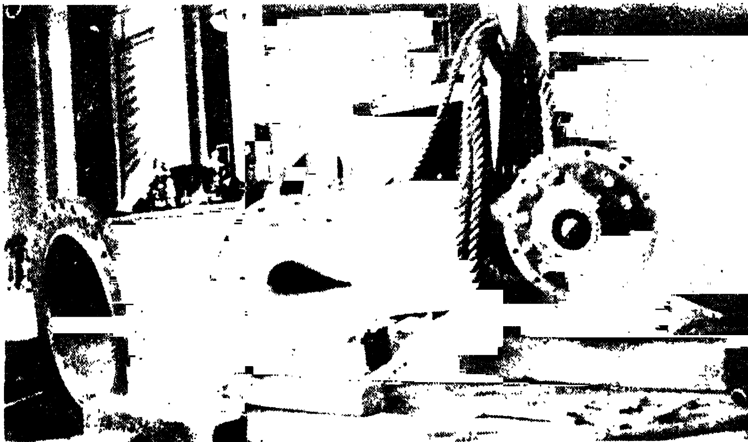
There are three different types of Pelton wheels at Mahora—one type is the type used in the four main generators, the second is for the exciter generators and the third type is for the motor generator set.

All these wheels are in single castings. In one of them the buckets have also been cast with the frame, while in the others the buckets are separate and are attached to the wheels by two bolts each.

The Pelton wheel is very simple. It can be said that it is the simplest type of a prime mover. The buckets are nothing but hemispherical cups and are divided by a ridge in the centre, which deflects the jet.

All the four Pelton wheels are of the horizontal type.

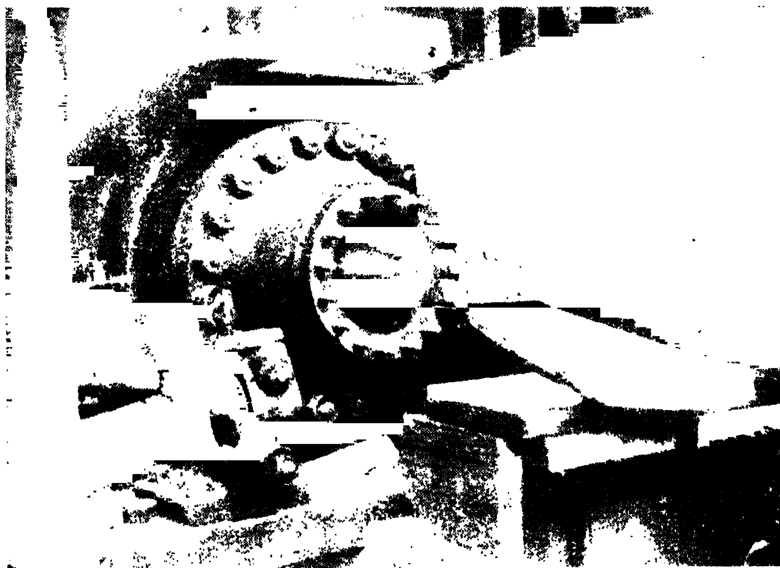
Fig. 10.



Elbow Tube of Pelton wheel showing the two needle-valves at Mahora.

There are two jets of water impinging upon the buckets of each wheel. Formerly multiple nozzle wheels were pretty common but recent practice in modern power-houses is to use only one nozzle in general. Certain conditions may call for two nozzles and in this way the power can be practically doubled. The jets are placed approximately at right angles in accordance with the general practice. To avoid the splash from one jet affecting the other and thereby reducing the efficiency, it is usually preferable to mount two single jet wheels side by side on the same shaft.

Fig. 11.



The nozzle of turbine, the neeule valve and the bearing for the deflector to be put.

At Mahora the nozzles are circular, with an axial needle or spear for regulating the size of the jet. The circular shape of jet suffers the least windage loss. The diameter of the jet is 6·5 inches. The maximum practicable diameter of a water-jet is 12 inches. For high efficiencies the diameter of the pitch circle of the buckets should not be less than about 12 times the diameter of the jet. In our case the ratio is about 9. The maximum specific speed obtained from a single runner and jet is limited to 5. The maximum speed obtained from a single jet is

$$n = \frac{5 \times (\text{head})^{\frac{5}{4}}}{\sqrt{(\text{H.P.})}}$$

In our case the head of water available being 400 ft., the maximum speed.

$$n = \frac{5 \times (400)^{\frac{5}{4}}}{\sqrt{\frac{9,000,000}{746 \times 10}}} = \text{about } 100 \text{ R. P. M.}$$

By using two jets on the wheel the higher speed of 500 R. P. M. has been obtained.

Fig. 12

Deflector Body.

Shaft.

Deflector

Sectional View of Nozzle.

Liner, C.I.

Removable Bearing of the Deflector Shaft.

4, 1" Thick webs.

Component parts of the nozzle of the Pelton wheel at Mahora.

To face page 463.

Size of jet :—This depends upon the head and power to be delivered. The quantity of water available is already ascertained. The pipe line losses from the source of supply to the nozzle can be computed.

Velocity $V_1 = 0{\cdot}96 \sqrt{2gh}$, where h is the net head at the nozzle in feet.

$$Q = V_1F, \text{ and } F = \frac{Q}{V_1} = \frac{\pi d^2}{4}.$$

$$\therefore \text{ the diameter d of the jet } = \sqrt{\frac{4Q}{\pi V_1}} \text{ ft.}$$

$$\text{or, } d_1 = 4{\cdot}88 \sqrt{\frac{Q}{\sqrt{h}}} \text{ ins.}$$

$$\text{But H. P.} = \frac{Qh}{11} \text{ (for 80 \% efficiency).}$$

$$d_1 = 16{\cdot}2 \sqrt{\frac{H.P.}{h^{\frac{3}{2}}}} \text{ ins.}$$

Diameter of a wheel :—This means the diameter of the circle to which the axis of the jet is a tangent. It is limited by the fact that the most efficient velocity of the bucket (mid point) is from 0·45 to 0·47 times the velocity of the jet, or taking 0·46 as the average, the linear velocity of the bucket $V = 0{\cdot}96 \sqrt{2g}\,H \times 0{\cdot}46 = \frac{\pi d n}{60}$, where n = R. P. M.

and d = diameter in feet.

$$\therefore d = \frac{67{\cdot}5 \sqrt{h}}{n} \text{ feet.}$$

The minimum diameter is fixed by the condition that it must be at least 10 times as great as the diameter of the jet ; where this condition conflicts with that for best efficiency, the stream must be divided into two or more jets.

Needle Valves :—This system consists of a nozzle body in which is inserted a concentric tapered needle. By means of this needle, which is manually controlled for this type of nozzle, the jet area is adjusted intermittently to correspond to either the stream flow or the maximum anticipated load likely to be carried within a certain time limit. *Vide* Figs. 10, 11 and 12.

The nozzles and the needle valves are to be changed occasionally owing to the fact that they were out because of the constant flow of water at a high velocity through them. Repairs to these are carried out in the workshops adjoining the power-house. For the worn out nozzle, *a liner, shown,* is fitted on after turning the inside of the nozzle to a diameter equal to that of the outside of the liner. The liner, then, is fixed to the body, by four studs. The needle valves, too, are repaired in a like manner. The tapering portion is of gun-metal, screwed on, in two parts, to a rod which is regulated with a hand-wheel. Only the gun-metal portions are changed (*vide* Fig. 12).

Buckets :—The buckets, as mentioned before, are of the elliptical shape which have superseded the original rectangular section. In the case of the rectangular buckets the sharp curves and corners of this type cause an appreciable loss in eddy formation, and tests show that the efficiency obtained with the modern form of bucket is from 6 to 10% greater than the older form.

The dividing ridge (on splitter) cuts the jet in half and owing to the form of bucket, these two halves travel right round the side thereof, where the spent-water issues at nearly 180° from its original direction. Though theoretically it should issue at exactly 180° from its original direction for getting out maximum efficiency, it cannot do so in practice owing to the fact that it would foul the oncoming bucket In practice, the angle is limited to a maximum of 165°.

The friction loss in the buckets increases with the wetted area and to reduce this the number of buckets should be as small as is consistent with continuous impact ; while they should be made no larger than is necessary to give the required change of direction with easy curves and without shocks. The surface should be as smooth and well-finished as possible.

As already stated, for minimum loss the buckets must be as few as possible and consistent with the jet being wholly intercepted for all bucket positions, so that the entering bucket may entirely intercept the jet before the leaving bucket begins to free itself. From these considerations the following formula is evolved :—

$$n = \frac{\pi}{\sqrt{1 - \frac{(r + t/2)^2}{R^2}}}$$

where, n = minimum possible number of buckets.

R = the extreme outer radius over the receiving edges of the buckets.

r = pitch circle radius.

t = thickness or diameter of jet.

In our case, n should be $= \dfrac{R\ \pi}{\sqrt{R^2 - (r + t/2)^2}}$

$$= \frac{26 \times 3{\cdot}14}{\sqrt{26^2 - \left(19 + \frac{6{\cdot}5}{2}\right)^2}} = 15{\cdot}8$$

and we have got 16.

By using the general formula, $n = R\sqrt{r/t}$, where R ranges from 7 to 8 as the wheel diameter decreases from 6 to 3 ft. By substituting the actual values of r and t, and taking R = 8, we get the value of n to be 14. A little additional overlap is usually given to allow for any slight variation in the usual position of the jet.

By varying the general formula $n = K\sqrt{r/t}$, where K ranges from 7 to 8, as the wheel diameter decreases from 6′ to 3′. By substituting the actual values of r and t and taking K = 8, we get the value of n to be 14, a little additional overlap is given to allow for any slight variation in the usual position of the jet.

In modern practice the width of the buckets is between 3 to 5 times the diameter of the jet impinging on the buckets, the ratio diminishing as the size of the jet increases. The ratio taken here is 3·5.

When the dividing ridges of the buckets are straight in profile, these are not fixed radially but are inclined backwards from the direction of rotation at such an angle as to give normal incidence on the first impact of the jet. If placed radially, the jet would be deflected into the rim of the wheel during the first half period of impact and would tend to produce serious inefficiency. It has been found that on buckets having a straight profiles the maximum efficiency is obtained when the ridge is at right angles to the jet at the mean position between the beginning and end of the full jet. In the wheels under consideration the buckets have been fixed inclined 30° backwards from the radius.

The buckets are fixed on to the periphery of the cast steel disc by forging the stem into holes made in the rim to receive them. The buckets are of forged steel and look quite rough even when they are new. Owing to the high velocity of jet-water the buckets have to withstand a heavy hammering many times per second and the centrifugal forces in them may come up to a big amount.

Bearings :—These are of two kinds : (1) Standard shaft bearing; (2) Thurst bearing. In vertical wheels the shaft bearing simply keeps the runner and the shaft central with the surrounding guide vanes and casing, the weight of the rotating parts being carried on a thurst bearing, which is usually placed at the top of the generator and supports not only the shaft and runners but also the weight of the generator rotor.

In horizontal units the shaft bearings hold the rotating parts centrally, and also support them.

Shaft bearings are either (1) babbited bearings, arranged for oil lubrication and used for all bearings that are submerged. This arrangement prevents the entrance of the grit or silt to the bearings, which would ruin them ; (2) Bearings made of lignum-vitæ. Lignum-vitæ can continue to work after a considerable amount of abrasion, and, for this reason, has been generally adopted for journals under water.

Babbited bearings, if accessible, are ring oiling, otherwise they are lubricated by forced feed. All large lubricated bearings are water-cooled.

Lignum-vitæ bearings may have forced feed grease lubrication, if required.

Vertical babbited bearings are best adopted for positions between turbine and generator, if any is required.

The bearing on the turbine is usually lignum-vitæ. It must be placed closer to the runner than an oil bearing, is simpler, requires less attention, and is equally efficient.

All the Pelton wheels at Mahora are of overhung type. This is a popular American design, its advantages being that it is a cheap compact form of construction which effects a considerable saving of floor space. Each wheel is mounted on one end of the generator shaft, one of the bearings being in between the generator and the turbine and the other being on the other side of the generator. No extra out-board bearing is provided. The bearing between the turbine and generator is a water-cooled one. There are five brass rings, each in four parts, connected together by screws, and as these rings rotate with the shaft, the oil is brought to the surface of the shaft due to the dashing effect. The bearing surface is only at the bottom and is of cast iron with half an inch of white metal lining. These rest on the body of the bearing and the white lining is such that on one side there is a longitudinal groove so as to allow the oil, from the oil chamber, brought up by the rotating rings, to come to the bearing surface. Through the oil chamber there are four pipes going in a cross-wise direction and connected outside, through

which water circulates bringing about the cooling effect, the supply being from the bottom jet and this water after circulation returns to the turbine and joins the tail race. The bearing is also provided with a glass gauge to indicate the level of oil in the oil well.

Housing of the Pelton wheels :—The lower part is made of iron castings and the upper housing of steel plates riveted into a cast iron frame. This type of housing eliminates all danger of breakage. Where the shaft of the runner passes through the sides of the housing, water leakage is prevented by a centrifugal disc and water guard, which device ensures a frictionless packing.

*Water Turbines for Shannon Generating Station, Punjab

A hydro-electric plant operating under an exceptionally high head is now nearly completed for the Shannon generating station of the Uhl River project, Punjab, India, and we give below a short account of the water turbines for it, illustrated by Figs. 13, 14, 15, and 16. The turbines are being supplied by Messrs. Boving and Company, Limited, 56, Kingsway, London, W. C. 2, and are of the *Pelton overhung type*, four in number, with single runners and single jets. The effective head, when the dam of the storage reservoir has been raised to its full height, will be 2,001 ft., but at first the machines will work at an effective head of 1,668 ft. when they will each continuously develop 17,000 brake-horse-power when running at the normal speed of 428·5 R. P. M. No more power is, however, to be developed, even at the maximum head, and the system of governing and control has been designed to ensure steady speed under whatever conditions that may be imposed. A general view of one of the sets in the maker's shops is given in Fig. 13 and of one of the runners in Fig. 14. These are 78¾ ins. (2,000 mm.) in diameter, with 22 buckets cast in pairs, and are claimed to be the most powerful Pelton runners so far built in this country. One of the double buckets is shown in Fig. 15.

The disc of the runner is of forged steel, bolted to a flange on the generator shaft, the overhang from the centre of the disc to the edge of the bearing being only 39⅜ ins. The shaft is 21 ins. in diameter, and the weight of the complete runner is 7 tons. The buckets are 23⅝ ins. (600 mm.) wide inside, and are made of stainless steel by Messrs. Thos. Firth and John Brown, Ltd.,

* By courtesy of "The Engineering," 22nd January, 1932.

Fig. 13.



A view of the generating set and the governor.

Sheffield. The bowls are all carefully ground and polished, and great pains have been taken to ensure that the centre *fins* lie all in one plane circumferentially. Each pair is attached to the disc by three tapered bolts, fitted with nut locking plates. Additional security is given by spot welding the nuts to the bolts at one point. The material or the buckets gave excellent results under test, showing an ultimate tensile strength of 50·4 tons per sq. inch, and yield point of 39·6 tons. The elongation was 22·5 per cent. with a reduction of an area 42 per cent. A $\frac{3}{4}$ in. by $\frac{3}{8}$ in. by $4\frac{1}{2}$ ins. test-piece is bent through 180 degs. without showing any sign of fracture. The casting of the buckets in pairs introduced difficulties in moulding but remarkably good and uniform results have been obtained ; only one or two very small pieces of metal having to be attached to the discs to secure accurate balancing. This operation was performed at the Rugby Works of Messrs. The British Thomson-Houston Company, Limited, the makers of the generators, and the runners were afterwards tested along with the corresponding generator rotors for five minutes at a speed of 860 R. P. M., which represents the highest possible *runaway speed* that can be obtained under the maximum head.

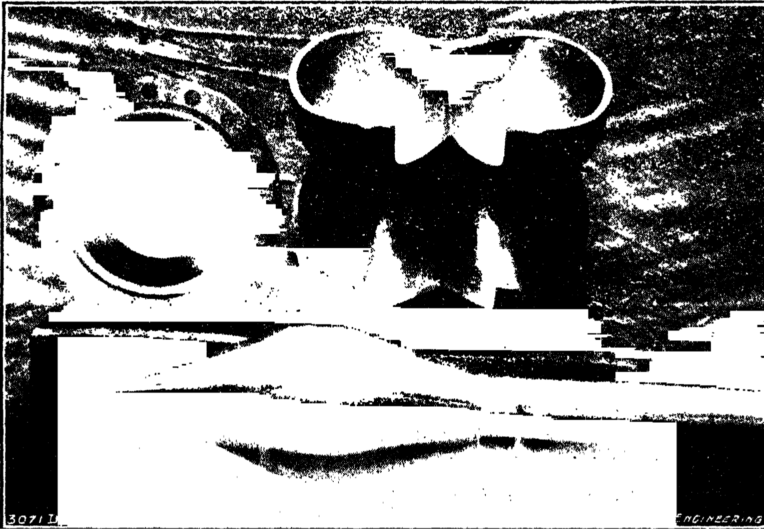
Fig. 14.



Runner of the Pelton wheel.

A needle and an inlet nozzle ring are shown in Fig. 16, but

Fig. 15.



Bucket of the Pelton wheel

in this connection reference should also be made to Fig. 18*b*. From the latter figure it will be seen that the tip of the needle is separate from the body, so that it is renewable. This tip is made of forged stainless steel, the bodies being of cast stainless steel of the same grade. The part of the needle spindle inside the inlet bend is of a different grade of stainless steel, giving satisfactory working in the guide bushes. The spindle is, moreover, protected from damage by a tube of galvanised mild steel. The inlet bends and nozzles are of cast steel, the nozzle being fitted with a renewable ring of a special grade of forged stainless steel. The maximum diameter of the jet is 8 ins. In order to permit of ready removal, the inlet bend is attached to the bedplate by a split ring. The deflectors are of cast stainless steel and have been machined so that the continuity of the jet is not interfered with in any way when they are out of action. The deflectors were also supplied by Messrs. Thos. Firth and John Brown, Ltd., and the other stainless steel parts by Messrs. Brown Bayley's Steel Works, Ltd., Sheffield. The casings of the turbines are of welded steel plate. The bedplate is of cast iron, but is extended downwards by a steel plate to form a protection to the turbine pit. Access to the pit is gained through a manhole, underneath which is a grid platform from which the nozzle, needle and runner can be conveniently inspected.

An interesting feature of the plant is the very complete system of control and safety devices provided in compliance with the requirements of the Punjab Public Works Department. Freedom from shock in the pipe line and very sensitive regulation is ensured by the firm's well-known needle and deflector system. On the load being reduced, the governor depresses the deflector and diverts the jet from the buckets, and, at the same time, and needle begins to close slowly under the action of springs, the necessary degree of retardation being determined by a timing valve in the port of an oil-dashpot servomotor. The needle moves forward and the jet, in consequence, steadily diminishes, the deflector being clear of the jet by the time the required degree of reduction is attained. Increase of load is provided for by the simultaneous opening of both needle and deflector, the rate of opening being such as ensures safety for the pipe line. The governing is effected by a sensitive pendulum contained in a pulley which is driven by a seamless belt from the generator shaft. The pulley shaft also carries a rotary oil pump for lubricating the mechanism. This pump can be used, in emergency, to control the deflector through a servomotor acting on a shaft carrying operating levers. In normal working the servomotor is supplied with pressure oil from a separate motor-driven pumping set which also supplies the needle servomotor. The shaft is also connected to the regulating valve of the needle servomotor. The whole of this gear is provided for each turbine, but, by means of oil ring-mains connecting all four turbines, of which it is expected one will be kept as a stand-by, failure of oil pressure on any one turbine is met by an immediate and automatic supply from the motor-driven pumping set on the stand-by turbine.

The governing and control gear is shown diagrammatically in Figs. 16*a* and 16*b*, the latter showing a section through the governor-head, but for the correct relative arrangement of the parts Fig. 13 should be referred to. It is impossible, in the space at our disposal, to describe, in detail, the whole of the mechanism illustrated, and the main features only will be dealt with. The needle-operating gear consists essentially of a servomotor cylinder A, the piston of which is actuated on the opening movement of the needle by oil pressure and on the closing movement by a spring. The oil, under a minimum pressure of about 150 lbs. per square inch, is admitted to the cylinder from the supply pipe B. It passes from this pipe to the outside of the regulating piston-valve C, which, in Fig. 6, is shown in the central, or floating, position. When opened,

however, by movement towards the left hand, the oil flows upwards past the locking valve D, normally open, and then downwards to the spring-loaded timing valve E, which is furnished with two rows of small ports. The pressure of the oil entering the valve uncovers both rows, and thus gives a comparatively quick movement to the needle when it is being opened. When the needle is being closed by pressure from the spring and the cylinder is acting as a dashpot, only one row of the small ports is uncovered, and the rate of travel is slower. These rates are expected to be, at their minimum, about 18 seconds for the opening movement and about 36 seconds for the closing movement. As the needle closes, the regulating valve C is moved to the right, and the oil is discharged from the cylinder round the smaller annulus of the valve, through the hole down its centre, and thence across the spring chamber to the return oil pipe F.

Other details of this part of the gear that may be noted are the hand pump G and the needle-locking valve D. The former is used to open the needle by hand in the event of the complete failure of the oil supply, and the latter, when failure occurs, automatically locks the needle in the position it was occupying at the moment of failure, by shutting off the cylinder to pressure or exhaust. It is actuated by pressure from the oil-supply pipe, which pressure normally keeps it open by a relay device. Cessation of pressure causes the controlling springs to close it. A pressure gauge is provided at H. The mechanism for operating the regulating valve C consists of a lever connected both to the needle spindle and to the governor gear, as shown in Fig. 6. The adjustable stop I prevents the governor from affecting the valve when the needle is required to remain open in a fixed position, a condition which will be obtained when the water has to be bye-passed without rotating the runner so as to provide a supply, if necessary, for other turbines, which may possibly be installed lower down on the water system.

The governor gear at the left hand of Fig. 16*b* may now be referred to. The pendulum J, of the evolvent type, is connected directly to a spindle passing through the rotary-pump shaft and terminating in a small valve K actuating the relay valve governing the servomotor. The pump is situateb at L and normally merely circulates the oil, but when any of the motor-driven sets are not in use, it delivers oil to a chamber surrounding the servomotor cylinder, which forms an air vessel and is indicated at M. The oil passes into the port at the right-hand side of the cylinder. This port is closed by the valve N when the needle and deflector

are under hand-control, and opens into an annular space round the relay valve sleeve O. The sleeve can be moved in the direction of its axis to alter the position of the ports in it relatively to the valve. The annular space communicates with another round the relay piston-valve P, which valve, it will be appreciated, is traversed laterally by the movement of the pendulum. The valve, as shown in the figure, covers a port leading to the servomotor Q. As it moves to the left, it admits the oil to the differential piston of the servomotor and produces the down stroke, the area on the underside of the piston, also exposed to valve, moves to the right, the piston is opened to exhaust by way of the port on the left hand of the cylinder, and upward movement then takes place, the oil being discharged to the reservoir below, whence it flows to the return oil pipe. The servomotor piston is coupled to the rocking shaft R by a connecting rod and a lever. The shaft is not shown in its correct position in Fig. 16*b*, but can be identified in Fig. 13 from the rod transmitting the motion to the deflector and the supporting column on the main inlet pipe. The quadrantal indicator, which shows by two pointers the position of both the needle and the deflector on appropriate index scales, can also be seen in this figure.

Returning to the mechanism of the governor gear, it will be apparent, from Fig. 16*b*, that the rocking shaft carries a third lever connected by a vertical rod to a lever on a spindle in the governor casing. This spindle carries a lever S, Fig. 16*a* of bridge form, on one part of which is mounted a roller engaging with the plunger of a dashpot. The centre portion of the lever is attached by a connecting link to a leverlike projection on a nut engaging with a screw formed on the spindle of the sliding relay-valve sleeve. The oscillatory movement of the rocking shaft is thus transformed into a traverse of the valve sleeve, which, by altering their relative positions, provides a compensating movement of the ports and valve edges, and so eliminates any hunting action of the governor gear. The movement of the valve itself is also effected by the hand adjustable load-limiting device T. This lever T is, further, subject to movement by a throw-over device, which puts the valve into such a position that the needle closes when the wheel reaches the overspeed limit. The closing is effected primarily by the centrifugally actuated stop U, which is situated in a ring attached to the generator shaft. The running speed, at which the stop will emerge and come, is to contact with the switch V, which is determined by an adjustable spring. On contact being made, the circuit of the current energising the solenoid W is broken, and a

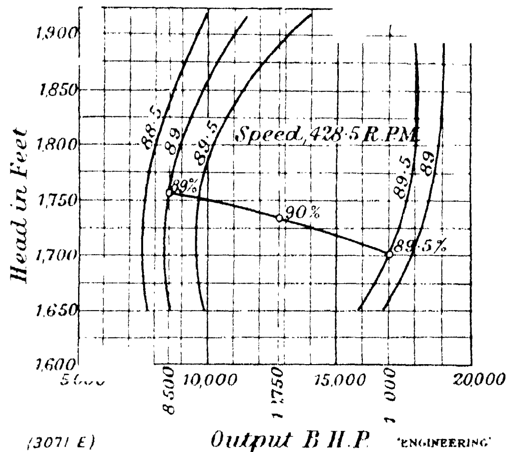
small valve admitting pressure oil to the plunger X is opposed to exhaust. The plunger is then released and the spring behind it forces it over, thus operating the lever T. The nut on the regulating sleeve is also controlled by a worm-wheel actuated by a hand-wheel Y, or by a motor Z, Fig. 16*a*. The former is used for altering the speed by hand, and the latter for altering it, or shutting down the machine altogether, from the switchboard. A contact switch for sounding an alarm on the failure of the oil pressure is seen at A As stated above, the needle is moved, in case of oil failure, by a hand-pump. The deflector, in the same emergency, is provided with both hand and motor-operating gear. Briefly, this consists of a mechanism connecting the hand-wheel b or the motor c, which has a chain drive to the rocking shaft through a dog clutch, the two halves of which are kept separated by the normal operation of oil pressure. Failure of oil pressure engages the clutch, and the rocking shaft may be oscillated as required by the hand-wheel or the motor.

The mechanism at the right of Fig. 16*a* is chiefly that for the automatic change over the pressure event of th from the oil pumping set of the turbine concerned to that of the turbine functioning as a stand-by set for the time being. The motor-driven rotary pump is seen at d. It draws from the return oil-reservoir and delivers to the air vessel. The delivery system is fitted with a non-return valve a, relief valve e, and an unloading valve f. The reservoir is provided with a cooling coil through which water is circulated, and has a filter at the filling point. Other filters are provided at several points in the smaller pipes of the oil system, and are indicated by the letter g. The letter h at other points denotes automatic oil-levelling device. The outlet pipe from the air vessel is connected to a two-way valve j, which has two branches one coupling it to the oil ring-main B, and the other coupling it to the governor servomotor air vessel. When the valve is in the position indicated in the figure, it is clear that the oil pumping set of the turbine is operating, but, when the valve is on the lower seat, this pumping set is cut out, and pressure oil is received from the ring main. When the pumping set is being used at a stand-by, it delivers oil to the pressure ring main through the lower of the two branches on valve j, by manipulation of the hand-operated valves indicated, the two-way valve being on the lower seat. A similar two-way valve l is provided for the return oil. The action of this valve will be clear from the figure. Both these two-way valves are automatically operated by a servomotor k. This is actuated by pressure

oil and is controlled by a relay valve fitted with a contact switch m to give an alarm on failure of the oil pressure. An air pressure system interconnecting the governor gear of the four turbines is provided, the pipe for which is seen at n.

As regards speed regulation, Messrs. Boving's guarantee is that the maximum *momentary rise in speed*, that will occur when the load is suddenly thrown off at 8,500 h. p., will amount to only $3\frac{1}{2}$ per cent. With loads of 12,750 h. p. and 17,000 h. p. the increase in speed is $5\frac{1}{4}$ per cent. and 7 per cent., respectively. The guaranteed efficiencies are shown in the diagram, Fig. 17. The three speeds just referred to are indicated in the diagram, from which it will be seen that the maximum efficiency of 90 per cent. is obtained with an output of 12,750 brake-horse-power and a head of about 1,735 ft. In order to bring the runner to rest promptly when the needle has been closed, a brake jet, with a stainless steel nozzle and playing on the back of the buckets, is provided. This is hand-controlled from the front side of the turbine, and the water supply comes from upstream of the main inlet valve.

Fig. 17.



The characteristics of the governor gear are that :—

(1) The speed of each turbine can be remote controlled from the switchboard.

(2) There is hand-control of both deflector and needle.

(3) The output can be limited by hand-control to the maximum of 17,000 brake-horse-power.

(4) Failure of oil pressure in one pumping set is immediately and automatically compensated for by the stand-by set.

(5) If the spare set should also fail, both needle and deflector are automatically locked in the positions occupied at the time of failure.

(6) When the deflector is so locked, it is motor-controlled from the switchboard.

(7) The deflector is closed in 5 seconds on the occurrence of overspeed, either by pressure oil or, should that have failed, by motor.

(8) Surplus water can be by-passed, if there is more than is required for the output.

(9) Adjustable speed difference between no-load and full-load is provided to ensure an even balance of load between units when generators are running in parallel.

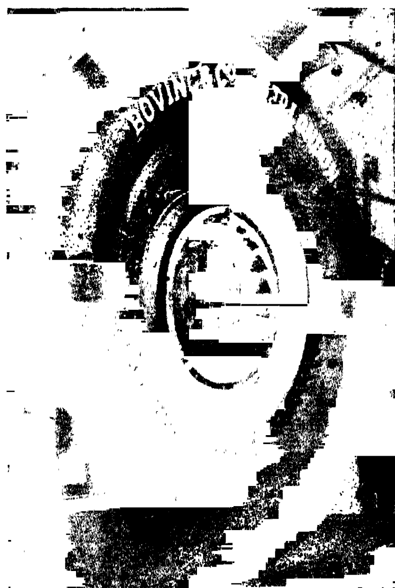
The whole of the turbine equipment has been designed by Messrs. Boving and Company, Limited, who are responsible for its performance. The turbines have been manufactured for them by Messrs. Markham and Company, Limited, Chesterfield, and the governors and pumping sets were supplied by Messrs. Boving's associated firm, Messrs. Verkstaden, Kristinehamn, Sweden. The inspection of the turbines has been carried out by Messrs. Preece, Cardew and Rider, 8, Queen Anne's Gate, London, S. W. 1.

Turbines in the Jammu Power-House (Francis)

Capacity—Nos. 1 and 2 are 186·5 kW. each ; No. 3—350 kW. and No. 4—437·5 kW.

All the four turbines in the Jammu power-house are *of the reaction* type with the Francis type rotor. They are made by J. Gordon & Co., England. There are *two runners in each turbine* fixed on the same shaft and the water enters radially inwards through the guide vanes. The moving vanes discharge the water axially towards the turbine and the discharged water from both the runners passes down to the tail-race through the draft tube at the middle of the turbine. The *working head of the installation is about 26 ft.* The actual head of water at inlet to the turbines is *13 ft.* normally and the suction head in the draft tube is *13 ft. The average flow of water in the channel is about 500 cu. secs.*

Fig 18.

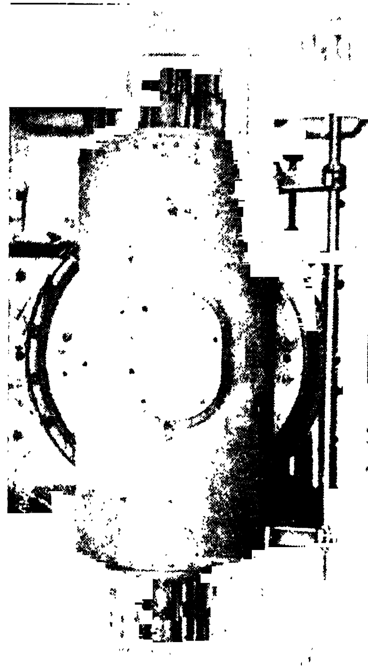


Reaction Turbine Inside View.

When all the four turbines in the power-house are running,

the tail-race water-level rises by quite a large amount, say, about 2 ft. This is due principally to the rising up of the bottom of the tail-race channel due to the deposition of mud and silt for a long time, the water of the channel being very dirty all throughout the year, and more so during the rains.

Fig 19.



Reaction Turbine, Power House, Jammu.

All the turbines are of the horizontal type. These multi-runner horizontal turbines are open to the objection that the gate mechanism is submerged and cannot be efficiently lubricated, while the entire mechanism is less accessible for inspection. The only advantage which can be claimed for this unit is its *higher speed.* The speed of turbines Nos. 1, 2 and 3 is 300 R. P. M. and that of the fourth is 275 R. P. M. The recent practice for low or medium-head installation is to have vertical type turbines and the guide vane ring is surrounded by a spiral volute-chamber from which the pressure water is delivered with uniform velocity around the entire periphery of the guide ring. As compared with an installation in an open forebay the modification of having the volute chamber makes the plant more efficient; and the guide vanes and the runner only are submerged. Horizontal turbines for very low heads are, however, necessarily set in open flumes or wheel pits.

The arrangements of having two or more runners on the same shaft has a number of disadvantages as compared with single runner. These are :—

(1) A separate series of guide vanes is required for each runner, all of which are submerged.

(2) Torsional deflection in the operating shaft renders it difficult to ensure equal gate openings on all runners.

(3) Cost of the sub-structure is usually greater than in the case of vertical single-runner turbines.

Fig. 20.



Turbine casing, Jammu.

(4) Owing to shock and interference between the discharged streams from the runners of a double-runner turbine, a greater proportion of the kinetic energy of discharge is lost than in the case of a single-runner turbine. This effect is increased by the impossibility of avoiding sudden changes of direction of flow in the water leaving the casing of double turbines. Since high velocity of discharge is a feature of a high specific speed turbine, this factor then becomes of special importance.

On the other hand, with a single-runner unit only one gate mechanism is necessary and this is outside the turbine casing and accessible for inspection at all times. Repairs to this can be carried out without dismantling the turbine. A long tappering draft tube, without any sudden changes of direction, can be used.

Horizontal turbines may be of a single or a double-discharge type, both admitting of an exposed gate mechanism. In the case of a single-discharge turbine there is only one common draft tube for the discharged water from the two runners, and in the other case there are two. The double-discharge has some advantage over the single, in that it is hydraulically against end-thrust. On the other hand, if it has a central discharge, *i.e.*, both runners discharging into a common draft tube, the draft tube conditions are not so favourable, unless the runners are spaced well apart. The turbine in Jammu power-house are of the single-discharge type.

In the case of a double-runner type of turbine there is very little end-thrust on the shaft because the thrusts of water on

the two runners very nearly balance each other. Any unbalanced pressure is taken up by thrust bearings. The thrust bearings which have been used in Jammu are of the simple collar type and are placed inside the generator room.

The turbines are fixed in the centre of the pits over the draft tube. They are mixed flow reaction type turbines.

In reaction turbines, the bearings bring in a lot of trouble. Previously the bearings were kept under water, but now this has been superseded by bearings which are out of contact of the water.

The first two turbines have got bearings inside the water, while the next two are covered round by iron sheeting of cylindrical form. The shaft is passed from one side of the cylinder, which is about 4 ft. in diameter. In this case the bearing is quite out of touch of the water and is lubricated by oil.

Fig. 21.



Double runner for the reaction turbine.

The disadvantages in having a bearing in the water are :—

(1) They are inaccessible, when necessary.

(2) Proper lubrication is impossible and hence they put in wooden bearings.

The water after proper guidance enters the runner and goes through the draft tube to the tail race from below the power-house floor.

Runners.—The runners are classified as the Francis and the propeller types. The Francis type may be subdivided into low, medium, and high head type according to the specific speed for which they are designed, while the propeller-type runners are designed for very high speeds. In Moody's propeller-type

runners, the flow is in a diagonal direction towards the axis of the line. It resembles a propeller with 3 to 8 vanes as compared to 14 to 24 vanes of the Francis type, the number depending upon the specific speed of the runner, the operating head and the problem of pitting. The openings here are twice as great, being free from the danger of clogging with foreign materials ; and the spacings of the intake trash racks are greater. This type is smaller and lighter, weighing one-half the Francis type of the same power. On account of higher speed, the cost of generator and foundation is also less in this type.

The runners made in one solid casting, with vanes cast in one pouring, together with hub and band, are much employed and are very strong. For very large sizes the runners are made in parts due to transport difficulties and troubles in casting. No shrouding band is necessary in propeller runners, and easily replaceable vanes bolted to a central hub can be used, *vide* Runner, at Jammu power-house.

Material of runner.—For moderate-sized turbines under low and medium heads cast iron runner in a single piece is generally used. The runners actually in use are of cast steel. Cast iron runners are preferable to steam runners as they are smoother than the steel ones, but where there is a chance of corrosion and wastage of the runner, due to solid material in suspension in the water, steel runners are preferable.

Correct design of vanes is more important than the material to avoid corrosion. Up to 200 ft. head, cast iron is quite satisfactory. For want of mechanical strength, in medium heads, cast steel is employed, specially when a large amount of trash is present in water. For high heads a special bronze alloy, resisting corrosion to a great extent or cast steel, is employed.

For moderate-sized turbines under low and medium heads cast iron runner in a single piece is generally used. The runners actually in use are of cast steel. Cast iron runners are preferable to steel runners as they are smoother than steel ones, but where there is a chance of corrosion and wastage of the runner due to the solid material in suspension in the water, steel runners are preferable.

Gibson gives the number of runners and the specific speed as follows.

Number of runners	3	4	5	6	8	10
Specific speed ...	122	106	94	86	75	67

Guide vanes.—The guide vanes are of cast steel as they are commonly made of. Under very high heads bronze guide vanes are used. The stems are cast in one piece with the vanes ; in large units only, the practice is to have the stems keyed to the vanes to facilitate their removal.

The stems are connected by levers all around the periphery and are coupled to a common regulating lever geared with a pinion rotated by the oil-pressure governor placed inside the generator room.

Draft-tubes and draft-heads :—A draft tube is simply a water and air-tight tube extending from the discharge of the turbine case down to tail water, and carried a sufficient depth below the water surface, to seal the bottom of the tube against entry of air, producing a suction action at discharge under a head equal to difference in level between runner and surface of tail water. If a draft-tube is provided, the wheel may be set at any height above the level of the water on the down stream side of the dam or tail water, up to a limit of 25 to 26 feet. The action of this column of water, in producing power at the water wheel, is the same as if the wheel itself were set down at the tail water level. The action may be described as similar to that of a siphon, the water exerting a suction proportional to the height of the column, the theoretical value of this height is 34 feet. At elevations above the sea level the available draft head diminished with the altitude. For maximum efficiency it should be well-designed and furnished by the turbine builder. It also converts the velocity head of the discharged water into pressure head. Hence, the main point to consider is that the velocity at exit to the draft tube should be least possible. Nowadays this is accomplished by tubes symmetrical about the turbine axle, though previously bent draft tubes were employed for the purpose.

The water leaving the runner whirls in the direction of rotation of the runner, the centre of the vertex coinciding with vertical axis of the draft-tube. Due to gyroscopic action, the vertex tends to remain in the same plane and serious eddies are likely to be set up by sudden turning of the axial component. This reduces the effective discharge area of the tube, as well as, there is a danger of water from the tail race, flowing back into the part of the horizontal discharge passage of the tube, causing serious eddy and outflow losses.

At present, Moody and White spreading hydraucone types of tubes of the symmetrical variety are much employed.

In the recent design in spreading type, the cone is carried clear up to the runner.

In Francis turbine the entrance is radial and the discharge axial. In propeller type the discharge as well as the entrance both are axial. The leaving water has a whirling motion around the turbine axis, and if permitted causes low pressures, cavitations, eddies, air pockets and dangerous librations.

The spreading draft-tube gradually decelerates the axial components, smoothly turning them in a radial direction, while the whirl component gets its velocity reduced to a minimum. By this turning of the flow in a direction away from the axis, the velocity of whirl diminishes inversely as the radius, the head diminishing inversely as the square of the radius, thus necessitating a moderate diversion of the water away from the axis for conversion of a large velocity head of the whirl into pressure head. After the deceleration of the flow in the annular passage of a free vortex, the water is collected in a spiral or double spiral passage and discharged horizontally into the tail race.

To avoid formation of a central cavity in the flowing stream due to whirling motion a central core, continuous with the runner, should be provided, if possible.

A correct draft-tube design is absolutely essential in order to obtain the maximum efficiency of a turbine as a whole. The fundamental principles underlying their design and construction are that the water shall leave the draft-tube with as small a velocity as possible, so that the maximum amount of kinetic energy is abstracted from the water. The velocity of the water in the tail race, furthermore, should be sufficient to prevent it from breaking up, and it is, therefore, necessary that the water emerging from the draft-tube must have at least a velocity equal to that in the tail race. In order to accomplish this, the draft-tube is constructed on a long radius. The section of the draft-tube is gradually increased to the tailrace, so as to gradually reduce the velocity of the water from the turbine to the tail race. If well-designed, the draft-tube enables a large proportion of the kinetic energy of discharge from the runner to be converted into pressure-head and so be utilised. The mean velocity of discharge from the runner varies from about $\cdot 2\sqrt{2gH}$ in a slow-speed turbine under a high head, to $\cdot 5\sqrt{2gH}$ or even more in high-speed turbines. Adopting the latter value the kinetic energy of discharge represents 25% of the total head, and it is essential that the large proportion of this energy be recovered.

Fig. 22.

Sec. I 4′ 11 7/8″

Sec. II 5′ 7½″

Sec. III 6′ 4″

Radius 10′ 3″

Sec. IV 7′ 0″

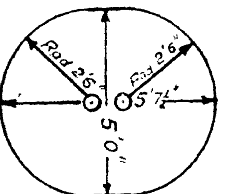
Radius 4′ 4½″

Sec. V 7′ 5¼″

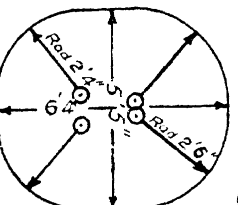
Sec. VI 8′ 0″



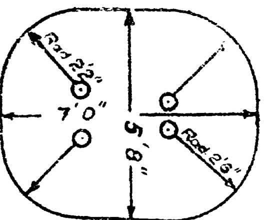
Sec I

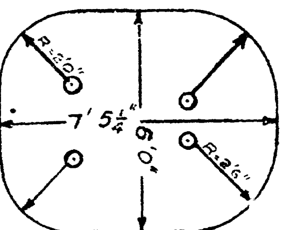


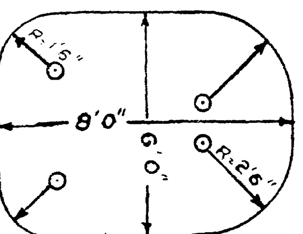
Sec. II



Sec III







Draft-tube at Siva.

With a parallel draft-tube discharging vertically into the tail race, the whole of the kinetic energy is lost. But if the velocity be gradually reduced by means of gradually increasing the diameter, the loss of energy is reduced.

The diameter of the draft-tube at entrance is about the same as that of the runner. This avoids all shocks at this point. The draft-tubes, here, are constructed of steel plates in accordance with the general practice. Draft-tubes, moulded in the concrete sub-structure, if not ruled out on the ground of expense, are preferable where circumstances permit, since it enables any desirable form of cross-section to be adopted. The common practice is to have an oblong section at the end, the longer axis being horizontal, which enables a comparatively shallow tail race to be used. With a steel-plate construction the conical circular form is almost essential.

Incidentally, a conical draft-tube tends to improve the speed regulation. With quick regulation on a falling load the inertia of the suction column tends to break it up and to produce a vacuum in the turbine casing. Any such separation in the column is followed by a reflux up the draft-tube which may give rise to severe water-hammer effect. Even a comparatively small change of load may, with a long tube, set up pulsations which are detrimental to steady running and which are reduced by the use of the conical draft-tubes.

Good draft-tube is fundamentally dependent upon the proper elevation of the turbine above tail water. The runner should be so located that the total draft-head at the top of the tube is well within the theoretical limits of a vacuum.

The reduction of the exit velocity of the water from the tube by gradually enlarging the cross-section at any point along the length of the tube is given by the equation—

$L^2V = A =$ constant for the area of the cross-section at any point along the length of the tube.

$L =$ length of the path from an origin to any point measured along the middle line of the tube.

$V =$ velocity of the water at any point.

Thus we find that the diameter of a round tube should increase uniformly from the draft chest to the lower end. The area increases the square of the diameter, so that the condition $L^2V =$ a constant is fulfilled. The final or discharge velocity V_e is not a fixed quantity. It is usually taken at a value such that the lost energy of discharge shall never exceed 1 per cent. of the energy due to the total head and shall preferably

be kept within 0·5 per cent. or $V_e = 0·4\sqrt{H}$ to $0·8\sqrt{H}$ is the range of the value of the velocity of exit. The principal loss in the draft-tube is, however, not due to exit velocity, but to eddy whirls and internal disturbances caused by the twisting motion of the water on leaving the wheel.

The diameter of a draft-tube at the bottom is limited by its length and by the diameter where it is attached to the draft chest. The maximum "mare" or rate of increase in diameter should not exceed 0·33 feet increase per foot length. The length of the tube is limited by the height of the turbine above the tail water. The draft-tube should not be longer than 20 feet.

Large draft-tubes are flattened at the bottom to avoid the cost of the excavation that would be required if the cross-section were maintained circular. The tubes are invariably made of concrete.

The effect of discharge velocity is to reduce the height that the runner may be set above tail water level and if V_3 be the velocity at the efflux from the draft-tube, with h_2 the elevation at the discharge ring above tail water level, we have

Vacuum at discharge ring $= h_2 + \frac{V_2^2 - V_3^2}{2g}$ where V_2 = velocity of water leaving the turbine, h_2 = the height of draft-tube setting above tail water elevation. The vacuum should not exceed about 28 feet for conservative practice.

Example. Find the height of the draft-tube setting above tail water elevation and the tube sectional area A at efflux when tail race velocity = 3 feet per sec.

Turbine efficiency at full load, 85 %.
Horse-power of the turbine = 584.
Head = 26 ft.
Discharge diameter = 120 ins.

Solution :—

$$\therefore \text{ Discharge} = \frac{584 \times 33{,}000}{26 \times 0·85 \times 62·5 \times 60} = 2{,}325 \text{ cu. sec.}$$

$$V_2 = \frac{2{,}325}{\frac{120^2\pi}{4 \times 144}} = 30 \text{ ft. per sec.}$$

$$A = 2{,}325/3 = 775 \text{ sq. ft.}$$

$$h_2 = 28 - \frac{30^2 - 3^2}{2g} = 14 \text{ ft.}$$

Care of Hydraulic Turbines :—Inspect occasionally the flume bearings, links, main shaft bearings, the fits between the runner and the shaft. In open flume turbines inspect everything once every four to six weeks. Systematically examine :

(1) The water passages, that is, the runner and guide valves ; see that there is no obstruction to the passage of water.

(2) Check the runner clearances with thickness gauges ; adjust the main shaft bearings to bring the runner to a central position.

(3) Inspect the gate mechanism for broken or work links and check the closing of the gates to see that all the gates close tightly.

(4) If regulating connections, *i.e.*, links, regulating shaft and bearings are fitted with grease cups or alemite fittings, force grease in each one of these.

(5) Inspect runner for pitting.

(6) Remove all foreign material from the flume.

(7) Examine the drain for leakage and see that everything is in proper condition before refilling the flume.

(8) In outside gate-mechanism turbines carefully examine that the lubrication is in due order.

Main turbine bearings of the water-lubricated lignum-vitæ type should be properly watched. There should be double screen so that one may be cleaned while the other is in service. This type of turbine should be drained and inspected once every two or three months when the following points should be noted :—

(1) The spiral casing, guide vanes and runner passages are all clear.

(2) See that there is no putting or erosion in guide vanes and runners. Keep records of accurate progress.

(3) With thickness gauges, inspect the runner clearness and at the same time inspect the turbine bearing for wear, making necessary adjustments, especially with the lignum-vitæ bearing.

(4) Close the guide vanes and test all vanes for accurate closing to prevent leaking when the unit is shut down.

(5) If eccentric pins for adjustments are provided in the gate mechanism, adjust the gates within two or three thousands for closing. If not so equipped, badly bent gates may be fitted with offset keys.

Turbines of the vertical type which are equipped with oil-lubricated or babbitted main bearings of the vertical type require very little attention for these parts, although they should be watched very carefully, and, if they are provided with thermometers, hourly readings of the bearing or oil temperature should be recorded. Horizontal bearings of the ring oiling type are very reliable and require little inspection, except to see that they do not lose oil, and that the oil level is maintained at the proper height. Vertical type bearings are usually equipped with some form of oil pump, and on large units these are installed in duplicate, with some automatic device, so that failure of one pump will start the second in operation. Frequent observation is essential, as failure of the pump will run the bearing in a few minutes. The level of the oil in the oil reservoir should be carefully watched, frequent inspection being made to make sure that water is not getting into the oil chamber as this will flood the oil out and may ruin the bearing. Babbitted bearings of the vertical type have been in operation for over ten years on some large units with no appreciable wear and very little loss of oil, although it is recommended that the oil be changed and entirely renewed once every year. Sometimes cooling coils are installed in the oil reservoir. In this case the flow of water must be watched carefully to guard against stoppage.

Usual efficiency.—The water wheel is usually designed so that the maximum efficiency occurs at about 80 per cent. gate opening. For tangential wheels the maximum efficiency is usually from 75 to 85 per cent. and for reaction turbines from 80 to 85 per cent.

Guide for Purchasers of Hydraulic Equipment :—When asking for quotations on hydraulic turbines and accessories, the engineer should give the manufacturer complete information regarding the physical characteristics of the site and the type of unit desired.

The physical characteristics are :—

(1) Maximum, minimum, and average static head.

(2) Fluctuation in head and tail water elevations.

(3) Length and diameter of pine line, if any, and preferably a profile of the same.

(4) The net head for which the turbines are to have the best efficiency.

(5) The shaft horse-power desired at a given net head.

(6) The speed at which the turbines are to operate.

The question of suitable and proper speed for the units should be definitely settled with the various water-wheel manufacturers, by asking them some time before calling for final prices, what speed they recommend for the given conditions. It may be desirable to ask for both turbine and generator, bids on two different speeds, so as to compare the cost of the units, together with their relative merits as to efficiency and the probability of pitting at the higher speed.

The specifications should include :—

(1) The type of wheel installation.
(2) The percentage of full gate power at which it is desired to have the point of best efficiency.
(3) Whether the most efficient type of unit is desired or whether efficiency is not of vital importance.
(4) Whether a reliable unit or a cheaper one, less reliable, is preferred.
(5) List of accessory equipment to be included, such as penstock valves, relief valves, steel draft-tubes, governors, etc.
(6) Imaginary limits to the turbine equipment within which the manufacturer is to furnish all necessary parts.
(7) Location of governor equipment.
(8) Whether structural steel supports are to be included.

These are usually furnished by the purchaser.

* **Specifications** :—The following data necessary for purchasing water wheel should be noted.

(1) (*a*) Type of turbine ; impulse or reaction wheels.
(*b*) Number of units.
(2) Horse-power of water wheels.
(3) kW. of generator.
(4) Net and gross head, quantity of water available.
(5) Open flume or closed flume.
(6) If closed flume, what is the number of pipes ?
(7) What kind of pipe ?—wooden stave, steel or concrete ?
(8) Diameter of pipes.
(9) Effective head (unless design of all-water passages to and from wheel is left to water wheel manufacturer).
(10) Head water elevation.
(11) Floor elevation.
(12) Tail water elevation.

* "Hand-book of Mechanical and Electrical Cost Data," by Halbert P. Gilleteo and Richard T. Dana.

(13) Head variable, if so, what is normal operating head ?
(14) If head is variable, what is the range of variation ?
(15) How important is power and economy at lowest head ?
(16) Speed of generator, if already decided.
(17) If speed of generator is not decided, name speeds which seem to purchaser most desirable, and ask recommendations. Name periodicity of A.C. system.
(18) Flywheel effect of generator.
(19) What speed regulation is desired for different load changes ?
(20) Will units run in parallel with other plants ? If so, give general characteristics of such plants.
(21) If running in parallel with other plants, can these plants be used to regulate the system ?
(22) What is the character of load factor ?
(23) What is the nature of water (silty or clear) ?
(24) What data for shipment of material is desired ?
(25) Advise, if it is expected that manufacturer shall furnish governor.
(26) Give sketches of power plant site.
(27) Give information as to what is to be expected in the way of guarantees.
(28) Method of regulating inlet ; in case of high heads, whether by spear, deflector, or deflecting nozzle.
(29) Give any other information, which, you think, would influence the design of the wheel.

Accessories :—Pressure gauge ; tachometer ; taper pipes from main pipe to nozzle ; main valve ; relief valve ; governor and connections ; spare parts.

Answer each point and note the following data supplied by manufacturers :—

(1) A check of the calculation on effective head.
(2) Horse-power guarantee at normal head.
(3) Guarantee at other heads, if head is variable.
(4) Speed guarantee, including runaway speed.
(5) Recommendation for best speed, if the same has not been determined.
(6) Speed regulation guarantees.
(7) Efficiency guarantees at full-load, $\frac{3}{4}$-load and $\frac{1}{2}$-load.

(8) Point of greatest efficiency of wheel and value of same in per cent.
(9) Efficiency guarantees for available head conditions.
(10) If water wheel manufacturer furnishes governor, give information as to the type, make, power required to operate the same, also what variation in speed will not be exceeded before the governor will begin to re-adjust gates to meet a change of load, either gradual or sudden.
(11) Complete drawing showing machinery proposed.
(12) Complete description of machinery proposed.
(13) Guarantee of durability.
(14) Guarantee of shipment.

For determination of size note the following :—

(1) the quantity of water to be delivered to the power plant,
(2) the head on different sections of the penstock,
(3) the length of the penstock,
(4) the nature of the application of the power,
(5) accessibility of the site,
(6) the design fixed upon for the wheels.

See if there is any coffer dam for controlling the flow and details of reservoir, abutments. Note the diversion work by which the water is carried from dam to the power station. Ascertain the flow determination and hydraulic gradient.

Determine relation of kW. cost to size of plant, and cost of development. Capital cost and annual charges.

Capacity — Capital — Interest — Depreciation — Oil—Care —Repairs and Operation—Total per H. P. per annum.

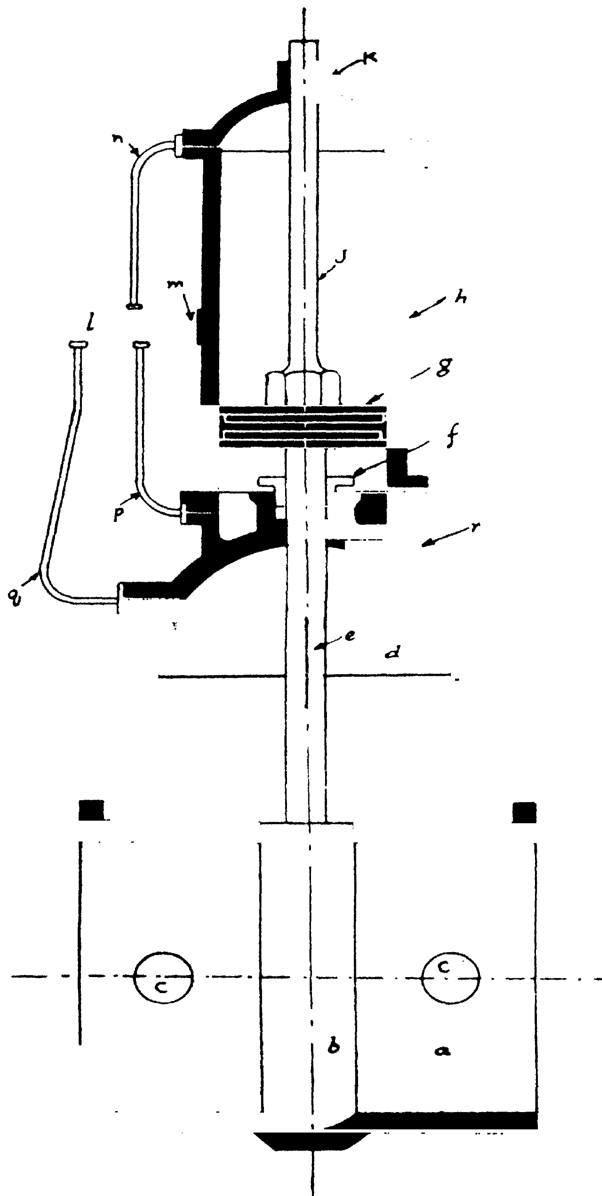
Units generated, Units sold or used, Rate—Tariff.

Load factor, Diversity factor, Plant factor and Power factor in different lines.

CHAPTER XII

REGULATING DEVICES

Fig. 1.



Main Valve

Main Valve:—The connection between the pipe line and each turbine incorporates a valve for the purpose of disconnecting the turbine for repairs. A sectional drawing of the type of valves installed at Sivasamudram is given in the Fig. 1. The valve is hydraulically operated but there is arrangement for hand operation also as an alternative in case of failure of power. The pressure water is taken direct from the pipe line. Referring to the lettering on the figure, *a* is the valve body bolted to the flanges of the adjacent pipes in the usual manner and is the valve door, *c* and *c'* are the branches for the bye-pass valve, enabling the water to pass *c* to *c'*, which, by filling both sides of the pipe, equalises the pressure. The valve chamber cover is shown at *d*, and *e* is the valve-spindle which passes through a stuffing box *f* to piston *g*; *j* is the piston tail rod which passes out through the gland *k* and serves the double purpose of guide and "tell-tale" to show how far the valve is open.

At *l* is a hydraulic control valve (not shown) which is bolted to box *m* of cylinder *h*. From valve *l* pressure water can be admitted to top of piston *via n*, or to the bottom thereof *via p*, according as it is required to open or close the valve. The opposite end exhaust through pipe *q*. When *q* is closed and *n* is open, the valve will be balanced in any position.

Balance Valve:—Balance valves are installed at the generating station Bhira, for the regulation of the supply of water to the turbines.

It has got the advantage that it occupies less space, because it can be installed in the pipe line itself; also it is cheaper. But it is not so reliable and when any repairs are to be done on the other side of the valve, whole of the pipe line has got to be drained through the drain valve, and the butterfly valves at the valve house have to be shut. But it ensures a smooth flow of water and less loss of head due to eddies, etc., at the entrance to the power-house.

The outer body consists of two parts both bolted to each other and to the pipe line at the two ends. The *inner body* is also in two parts connected by a brass bolt and nut. The inner body is cast with four wings tapering towards the outer end and having four holes at the bottom. The lower portion of the inner body has got a bye-pass which is opened by a bye-pass valve operated by a handle from the floor of the power-house.

A circular piston working in the space between the inner and the outer body opens and closes the main entrance of water in the inner body. The stroke of the piston is 12½".

Fig. 1 (a)

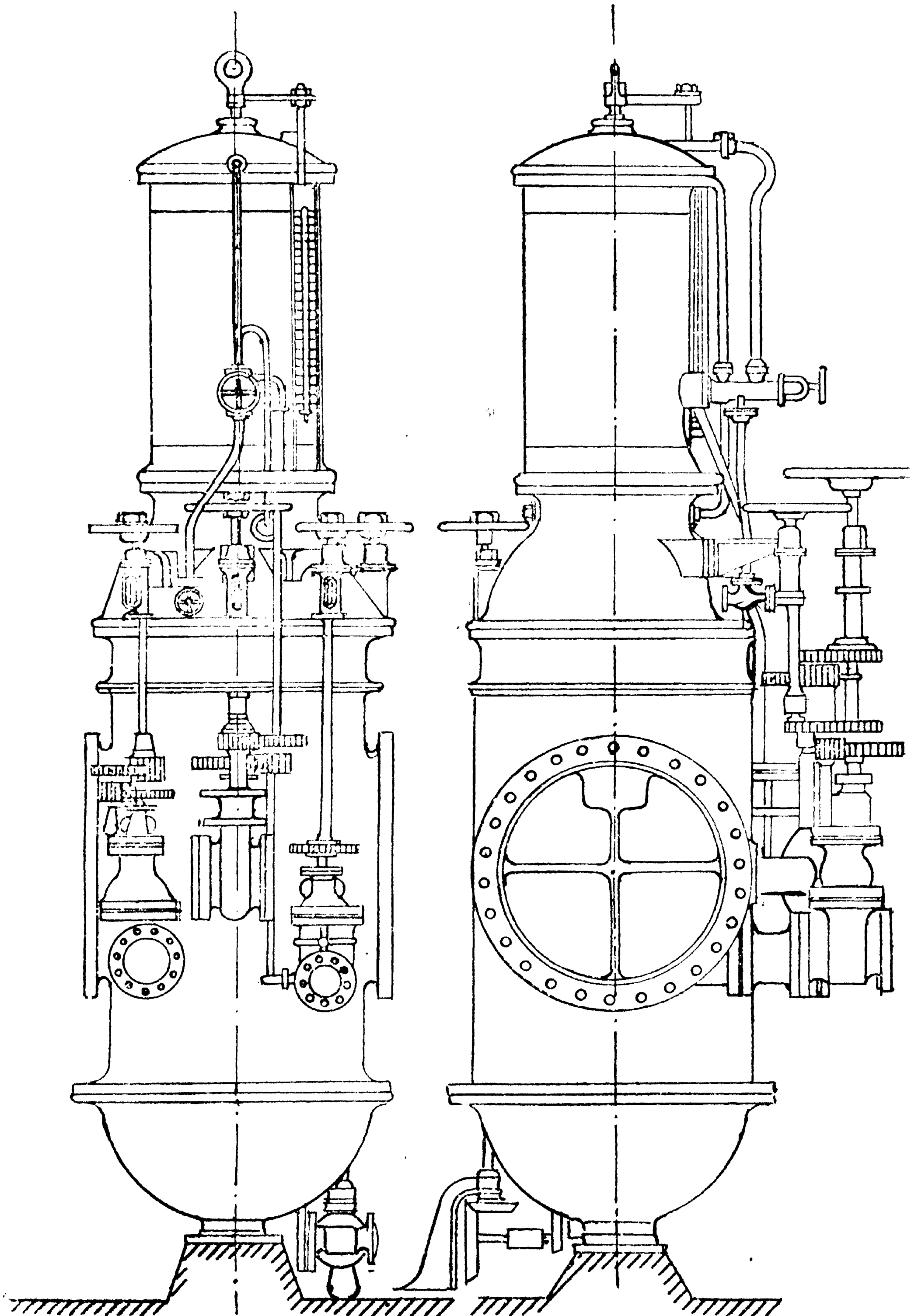


Fig. 1 (*b*)

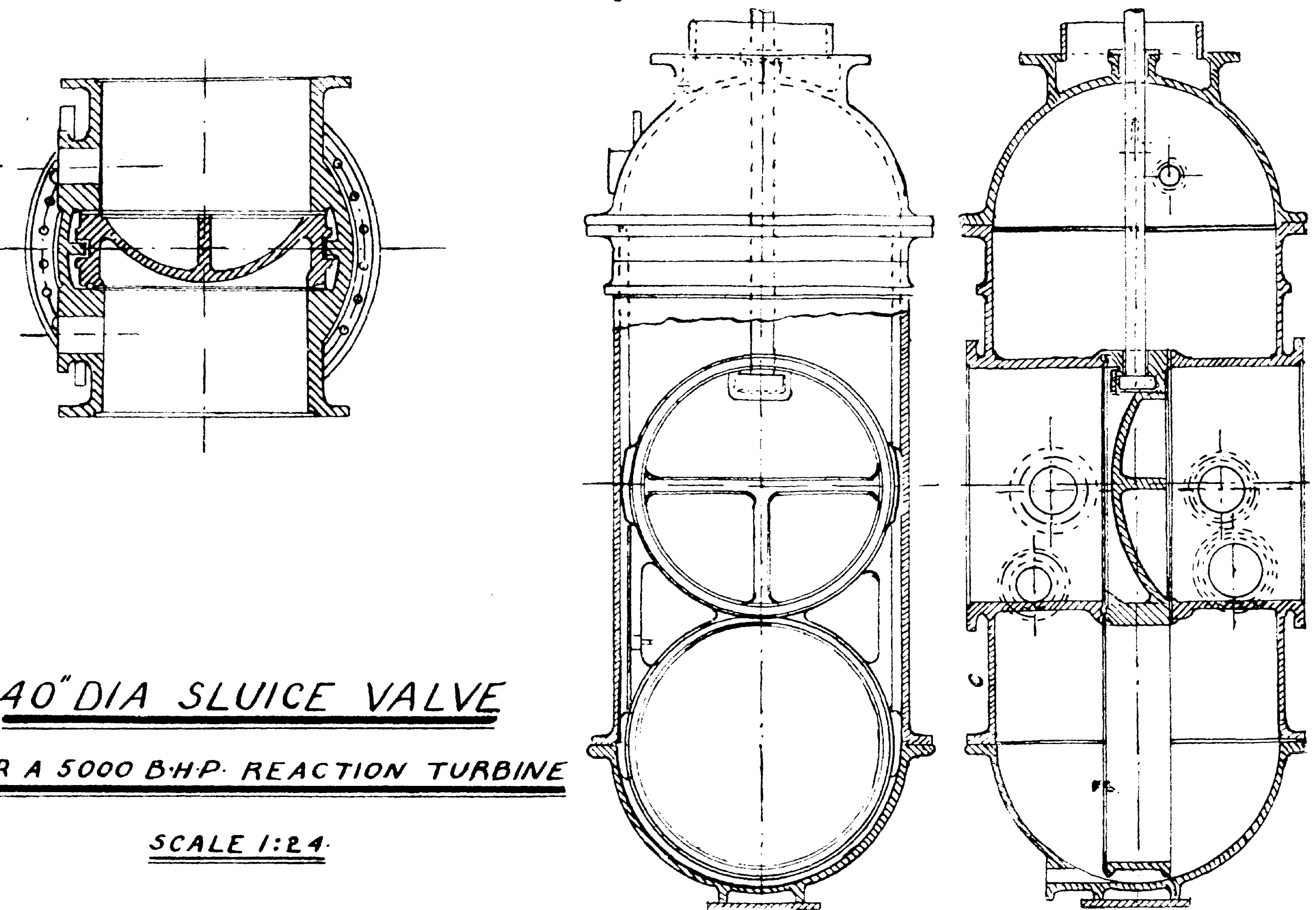
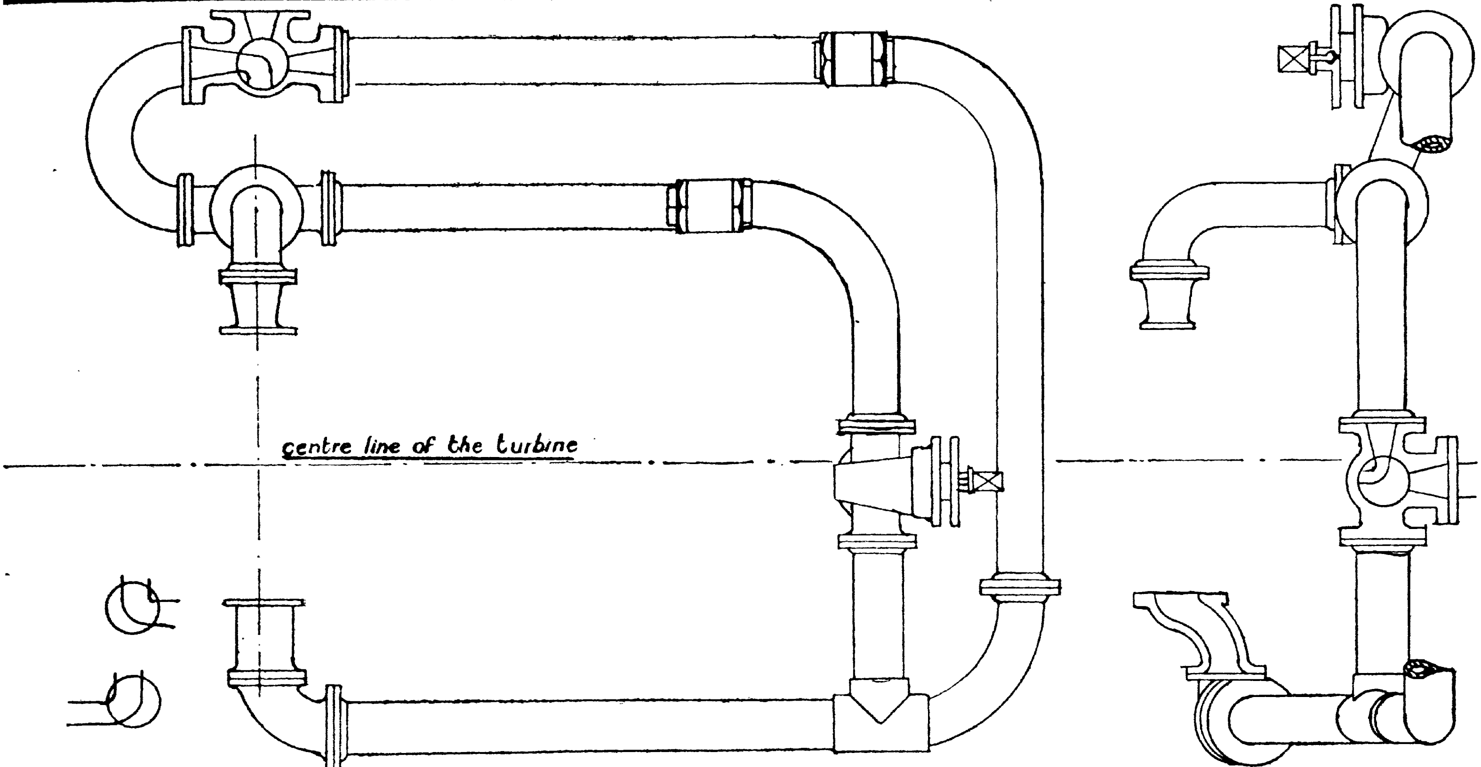


Fig. 2.



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The rubbing surfaces of the piston and the inner body are lined with staybrite liners to guard against rust and corrosion, etc. To make the working of the piston water-tight, leather rings are bolted to it at the outer end and also at the joint of the inner and outer bodies of the balance valve. These leather rings have got small rent holes, through which the oil under pressure enters the lower side of the rings, and keeps them tight fitting against the inner and outer bodies, ensuring water-tight working.

Bye-pass valves may be utilised to maintain constant velocity of the water in the pipe line by opening the bye-pass the same amount as the passage way through the water wheel gate is closed, and vice versa. Such valves are very wasteful of water, but their use is justified in certain cases where the stream is utilised for irrigation purpose. Deflecting nozzles used for impulse wheels belong to this class of devices. To conserve water the bye-pass may be arranged so as to close gradually after it has opened. This closing should be so gradual that no water hammer is produced in the penstock.

When the generator is to be started, the bye-pass valve is opened first Water at a pressure of 750 lbs./sq. in. enters the bye-pass, and then passing through the four holes in the inner body, balances the pressure on both sides, whence the name *balance valve.*

The oil under pressure is admitted in the opening chamber through the air hole. The oil pressure pushes the piston forward opening the main entrance of water. The oil on the other side of the piston is open to the exhaust through the other air hole. Similarly for closing the generator, the oil under pressure is admitted to the closing chamber of the piston which forces it backwards till the entrance of water is completely closed ; the oil in the opening chamber is now open to the exhaust through the air hole first mentioned

There is an indicator outside the balance valve, operated by water in a cylinder, connected to the opening chamber. It shows when the piston is open or closed.

The connections of oil pressure to the opening and closing sides of the piston and the exhaust are made through a pilot valve. This valve is operated by water pressure directly from the pipe line by a small copper tube. It has got a small slide valve operating on 3 parts. When the handle is pushed to the 'open' or 'shut' positions, the oil pressure is connected to the corresponding opening or closing chambers of the balance valve.

Connection is also taken directly for the pipe line by a copper tube to the Bristol's automatic recording gauge which records the pressure of water at the power-house and of the pipe line throughout the 24 hours. It is changed daily at 12 in the night for each machine.

The balance valves give quite satisfactory working, but for the leather rings which sometimes give trouble, and due to which it is not always safe to work on the other side with the balance valve closed and the pipe line full of water.

Relief Valve :—These are similar in purpose to the safety valves on a steam boiler. They allow all excess pressure to escape but do not restore energy when the pressure drops. In order to prevent the setting up of pressure waves the relief valves must be so constructed that they will close very slowly after they have opened.

There are many types, some depend altogether upon the pressure of the water column itself and others being opened by the same movement of the governor which closes the gates. The latter class of valves are on the whole preferable because they provide a passage through which the surplus kinetic energy of the water column may be discharged without waiting for a rise in pressure. The governor actuated valves are generally designed to close slowly after the initial opening movement.

Boving Co.'s Relief Valve :—The action of a typical automatic relief valve for turbines in the power station at Shiva Samudram is given here with reference to the diagram attached (Fig. 3).

The automatic relief valve is actuated by pressure water. This pressure water is taken from the turbine pipe on the up-stream side of the turbine main valve, so as to allow the shutting of the relief valve before the water is let into the turbine.

The pressure water is passed through a filter (A) before entering the servomotor of the relief valve. Between the filter and the servomotor cylinder is placed a perforated disc (B). The size of this perforation regulates the shutting time and opening time of the relief valve. The smaller the perforation the quicker the relief valve opens.

The regulating valve (C) is kept in "shut" position partly by water pressure in the cylinder (S) and partly by the weight (D) on the top of the dashpot. Before filling the turbine, care is taken that the regulating valves are easily done, so the stuffing box (E) is not screwed tight and the connecting links to the dashpot should be forced for any slight movement and should

never get jammed. The dashpot is filled with oil (same as used in turbine governor) up to 15 mm. from the cover, when the piston (G) is in top position. While filling with oil the perforated plug (H) is removed so as to get all the air away from below the piston of the dashpot. The dashpot has an air hole (J).

When the water has been let into the turbine casing, the turbine guide apparatus is kept shut and the action of the relief valve is tested. For this purpose the regulating valve (C) is screwed full down by means of the hand wheel (K). The relief valve will then be fully open. The regulating valve is screwed up again and the relief valve tested, whether following the same action with REGULAR speed. If this is not the case, the shutting box (E) must have been too tight or there is some jamming in the connecting links to the dashpot, or perhaps the weight (D) is not sufficient enough. If the relief valve is following too slowly and not causing a big pressure rise in the pipe line, the hole in the plug (H) of the dashpot should be increased. If it is following too quickly so that a big pressure rise can be noticed in the pipe line, the hole in the plug (H) should be reduced.

To be sure that the automatic relief valve is always in perfectly good condition, it is tested once in a week when the turbine is not running. Even if the turbine in running, this test is made by partly opening the relief valve. Two small rings (L and M) on the regulating valve spindle and on the valve casing are put to ascertain the stroke and good action of the relief valve.

When the relief valve is shut, these rings will be tightened *slightly* in their lowest position. When the relief valve is acting, these rings will slide upwards and will indicate afterwards the length of the stroke of the relief valve. If only the regulating valve ring (L) has been moved and not the one on the valve casing, the relief valve is not acting properly and has to be adjusted.

Both the regulating valve spindle and the large spindle of the relief valve are greased daily by means of the grease cups (N and O) and also the outside of the valve casing (P) which is moving through the cover.

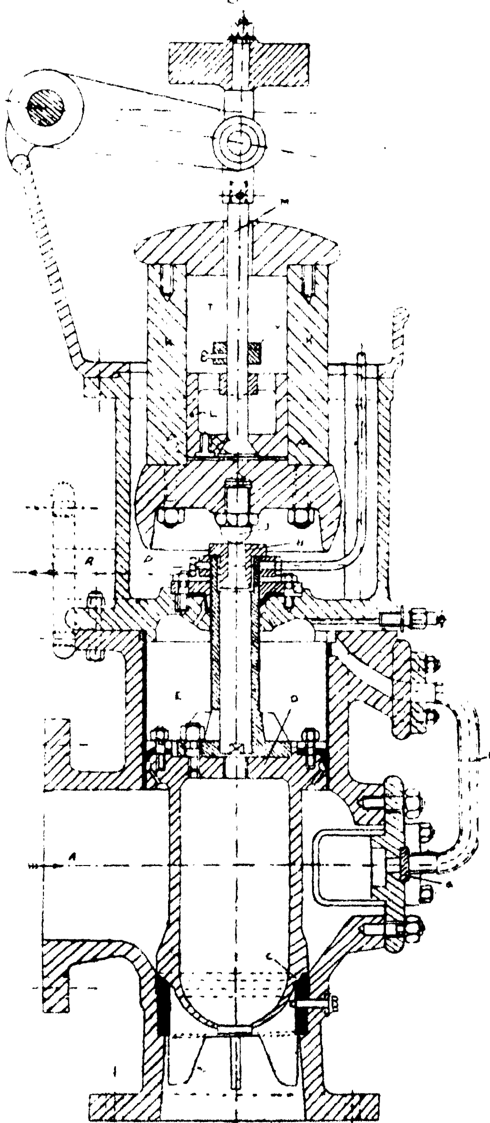
By opening the cock (R) and letting air into the discharge pipe, the noise of the relief valve, when acting, will be reduced to a considerable extent.

Principle and Action of the Relief Valve :—The pressure of the water both in the chamber (S) and over the valve (V) is same, but since the area of the piston (S) being larger than the area of the valve (V) there is always a more upward pressure tending to close the valve (V) and thereby shutting off the passage for water

through the relief valve, in spite of the direct water pressure from main pipe line over the valve (V). But when any change in the guide apparatus occurs such closing of the turbine gates, the regulating valve (C) opens and the water in chamber (S) is let out, hence the pressure is broken, thus allowing the valve (V) to open and relieve the pressure in the pipe line.

Escher Weiss & Co.'s Relief Valve :—The valve is shown in Fig. No. 4. The water enters through the branch A, the outlet B is normally closed by the valve C. The latter is in one piece with piston D, which moves in a cylinder. The space E above this piston is continually supplied with water under pressure from pipe F, at the entrance of which a filter and the small diaphragm G are fitted. From the space E an outlet leads through the hollow piston rod and the opening H, into the free space P above the cylinder to which an outlet branch R is connected, which leads to the tail race. The opening H is in the ordinary course closed by the valve head J. In this case the full pressure is obtained in space E, whereby the piston D and with it the valve C are continuously kept in their lower closed position.

Fig. 3



Standard relief valve, Escher Weiss & Co.

The valve J is fitted to the lower end of the dashpot K, which consists of a casing fitted with oil and designed as a balancing weight in which the piston L moves. This piston is provided with a diaphragm at its lower end through which the oil can pass from one side of the piston to the other side. The piston rod M is connected to the speed governor through the shaft S in

such a way that it is lifted up on the closing action of the governor taking place.

In the case of the load being thrown off the turbine, the closing movement of the governor occurring as a result, the relief valve works in the following manner :

The piston L is lifted up by the piston rod M, which moves upwards. As a result of the vacuum underneath piston L the whole dash-pot K, with the valve, is lifted up; the outlet H is opened and the pressure in space E falls considerably, because only a small quantity of water flows through the diaphragm G. The piston D is then lifted up by the water-pressure which operates on it continuously from below, and the valve C opens to an extent which corresponds to the closing movement of the turbine. Under the influence of the weight of dash-pot K, the oil is then forced into space T under the piston L, which, as a result of the small diaphragm, can only take place slowly. The dash-pot, with its valve J, falls correspondingly slowly, whereby the opening H is closed up to a point where the pressure in space E overcomes the pressure from below. From this movement downwards the piston D with valve C, following the movement of the dash-pot, moves towards, until the outlet is again closed.

The Governors

For a close-speed regulation in a hydraulic prime-mover under a varying load it is necessary that it is provided with a reliable governor, that the inertia of the rotating parts be sufficiently great and that the power-conduit be so designed or so equipped that the rise or fall in pressure in the turbine, following any probable change of load, is only a small fraction of the working head.

As regards the latter factor, the plant, which can be controlled most easily, is, that in which the turbine is installed in an open forebay fed directly from the supply canal. Here a demand for power is instantly met by an increased flow in the turbine, with the velocity corresponding to the working head ; in this case and also when the turbine gates are closed on a diminishing load, inertia effects are negligible. Both head and tail-race should be of ample dimensions, so that any fluctuation in flow may not cause an appreciable change in either level, while the approach channel should have easy curves and a well-finished surface inside, to reduce the possibility of any periodic wave formation.

The most generally used method for governing the speed of reaction turbines is by means of guide vanes, which change the amount of water supplied by simply altering the water passage. The vanes rotate about pivots and are fastened to a shifting ring by a link which is operated by pressure cylinders actuated by the governors.

The characteristics of the turbine have an important bearing on the speed regulation. Where the turbine efficiency falls off appreciably between three-fourths and full-load, the difficulty of close regulation over this range of loads is greatly increased, since an increase of load requires a greater additional supply of water than would be the case with a constant or increasing efficiency.

Owing to the inertia and considerable frictional resistances of the turbine gates, the centrifugal governor is not sufficiently powerful in itself to keep the required motion, and some form of relay becomes necessary. In earlier turbines various types of mechanical relays were used. Where the change of load is relatively small, this type fulfils the requirement, but for the more exacting requirement of electric generation it has proved inadequate. For such a purpose the pressure-operated relay provides the only satisfactory solution. Here the centrifugal governor operates a regulating valve which admits either oil or water under pressure to one side or the other of a piston in a relay cylinder, this piston being connected to and operating the gate mechanism. Plunger pumps or gear wheel pumps are generally used for producing the required pressure in the oil.

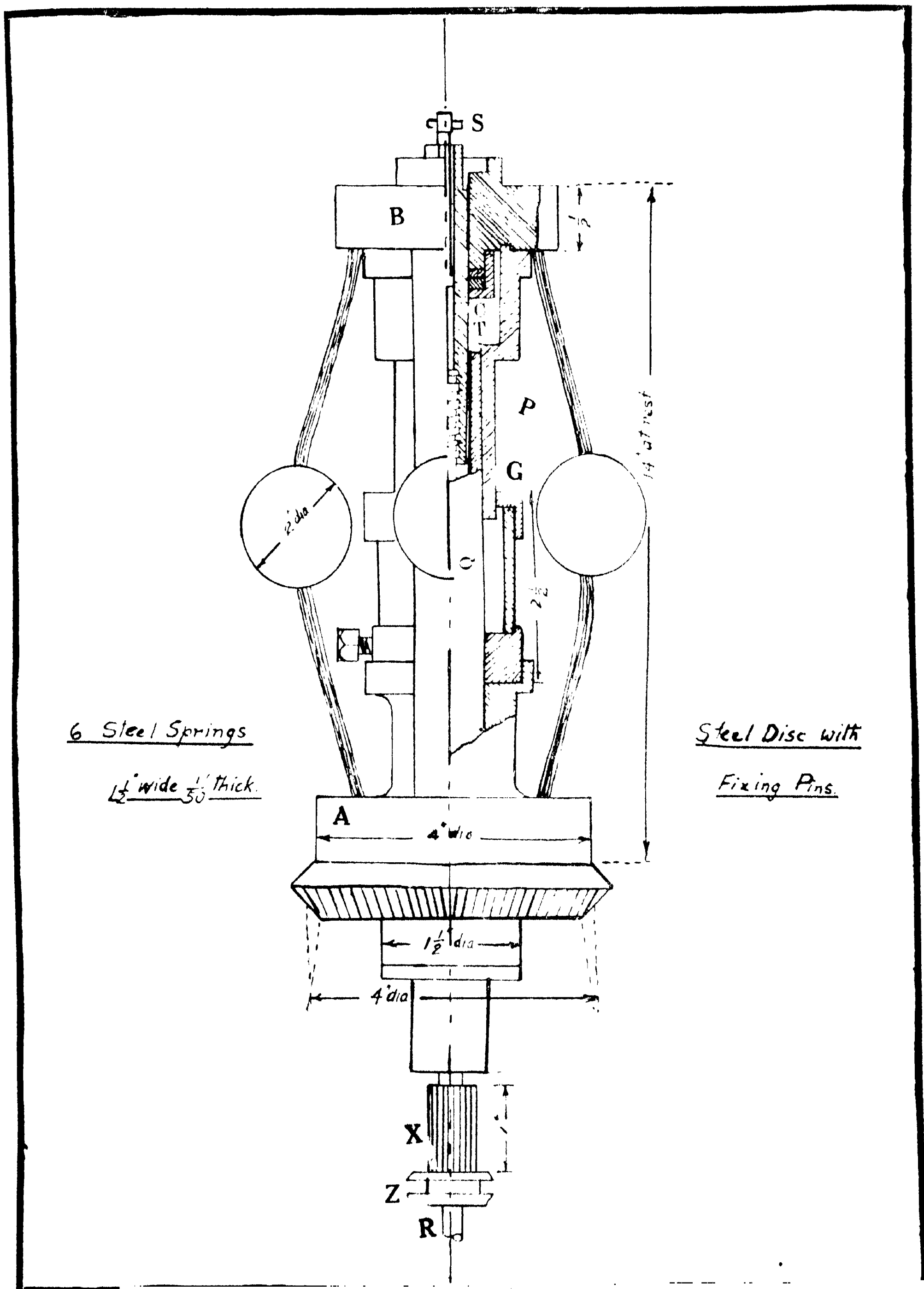
Governors at Jammu

For the four turbines in the Jammu power-house there are four separate governors. The governor of the 1st is of Boving & Co., England, second of them is made by the Lombard Governor Co., U.S.A., and the other two by Gilbert Gilkes & Co. of England.

Each of the Nos. 1 and 2 governors consists mainly of 6 different parts, *viz.*, the centrifugal governor, the relay or oil-distributor valve, the oil pumping arrangement, a reservoir for the oil, the pressure cylinder with its piston and a damping arrangement. The other two governors are also practically of the same form as Nos. 1 and 2 except in that they have got oil reservoirs in them.

In governors Nos. 1 and 2, the centrifugal part is of the flying ball type with the balls fixed on flat springs. Each of the two springs consists of 6 steel-plates, 1½″ wide, and about $\frac{1}{50}$th of an inch thick. These springs are firmly screwed to the steel

Fig. 4


S
B
C
T
P
G
Q
A
X
Z
R
6 Steel Springs
$1\frac{1}{2}$" wide $\frac{1}{30}$" thick.
Steel Disc with
Fixing Pins.
4" dia
$1\frac{1}{2}$" dia
4" dia
2" dia

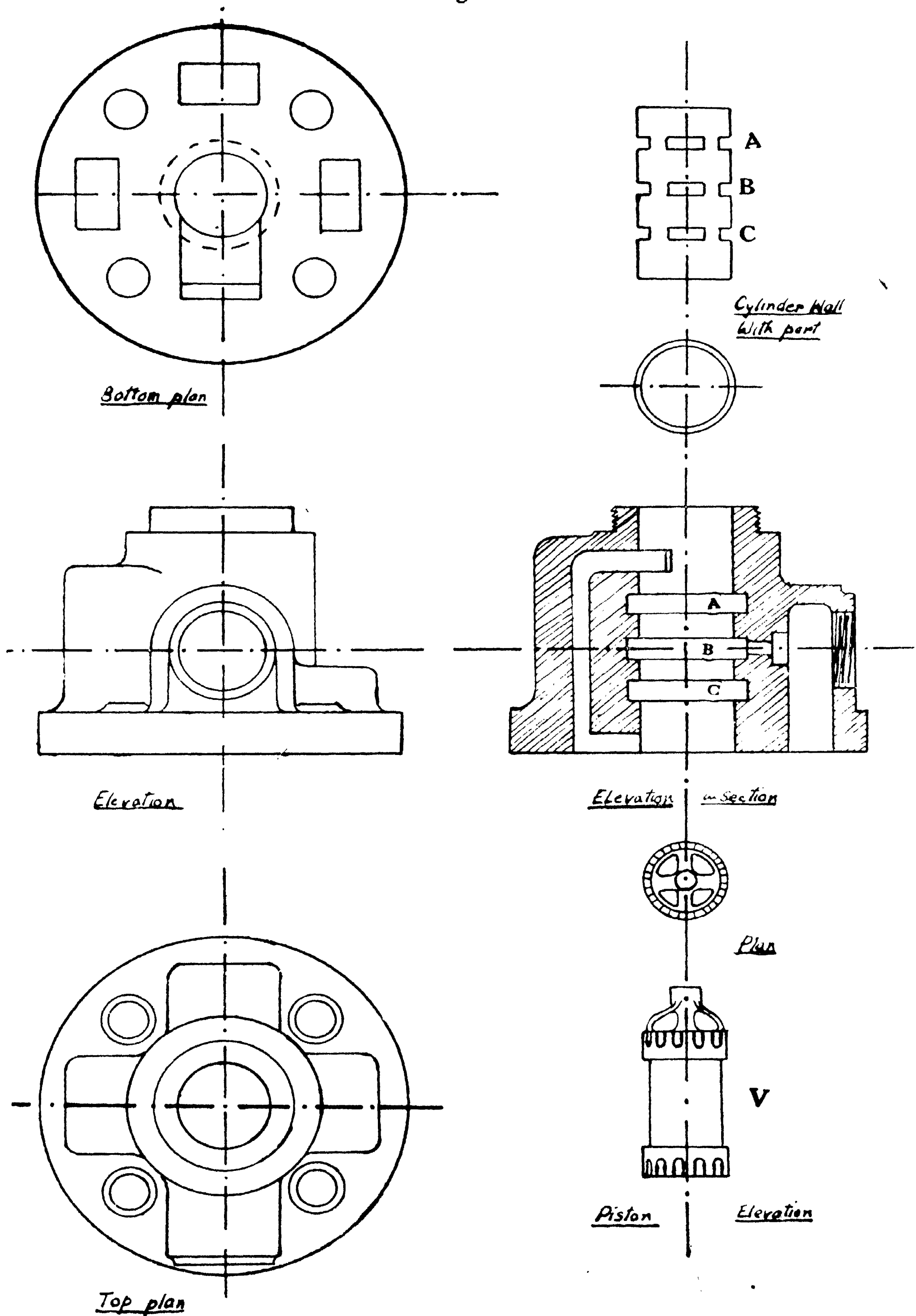
disc A attached to the bevel gear, as shown in the accompanying sketch. The upper ends of the springs are similarly screwed to the steel disc B. The bevel-gear, which drives the governor, is run off the turbine shaft through a belt. The average speed of the governor, as calculated from the pulley ratios, is 460 R.P.M. The rod R is attached to the relay piston which is placed just below the centrifugal governor, and *screwed* into the brass tube T. The thread of the screw is a square one and the pitch is 1", it being a triple-threaded screw. The rod S is attached to the tube T by a hinge. The rods S and R do not rotate while all the other parts of the governor rotate. The cup C is rigidly connected to the tube T and a projecting ring in the disc B revolves inside this cup on steel washers. This cup thus takes up the thrust of the ring B when the balls are revolving. The cup is always kept full of oil which is poured through a hole at the centre of the disc B. The brass sleeve P is attached to the disc B, and slides over the tube Q, which is latter attached to A, when the balls fly in or out. The tube G is a guide for the sleeve P in its motion up or down.

The composite rod RS can be lengthened or shortened at will by a hand adjustment or by revolving the gear X by means of the rack attached to the cylinder of the compensating mechanism, to be described later. The pulley Z rotates inside the two legs of a fork, which can be lifted up or down by a hand-lever. By pulling the lever up, the relay valve piston is lifted up and the pressure-oil is distributed to the pressure cylinder, in such a way as to gradually close the turbine down.

The relay or oil-distributor valve piston works in a cylinder with four port openings, as shown in the sketches attached. There is a lever inside the cylinder truly ground and the valve piston slides up or down inside this lever. The movement of the piston is very slight during the actual operation, one-eighth of an inch up or down being sufficient to move the turbine gates through a considerable amount.

In its mean position, the relay valve piston V (Fig. 5) occupies such a position that the ports A and C are closed by the ends of the piston. The middle portion of the piston is always kept in communication with the port B and it is this port through which the oil under pressure is admitted into the valve. The two sides of the piston in the pressure cylinder are connected to the ports A and C. When V travels up or down due to any variation in the speed of the turbine, either A or C is opened to the flow of oil enclosed in the middle portion of V. By the rush of the oil, the main piston is moved and the oil contained on the other side of the piston

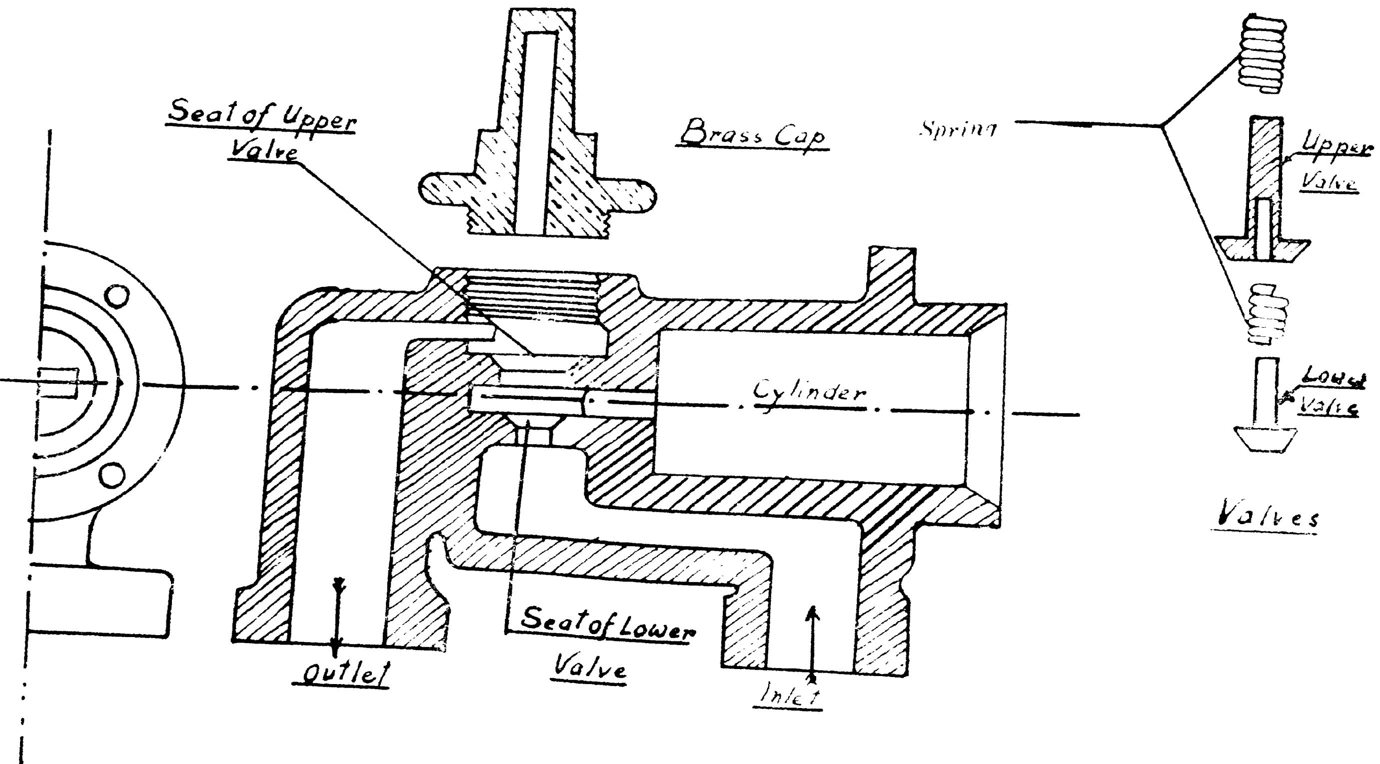
Fig. 5



Relay Valve and Cylinder.

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Fig. 6



Cylinder of Governor Oil Pump Reciprocating.

finds its way into the relay valve through the other port, and is then discharged to the basin below, to be pumped up again.

The oil pumps in the governors Nos. 1 and 2 are of the reciprocating type, the piston of which is worked from a cam shaft driven by a belt from the main shaft of the turbine, Fig. 6.

The reservoir for oil has got two compartments in it, one of which is to contain the oil under pressure and the other is to receive the discharged oil from the pressure cylinder. The former is provided with a pressure-gauge and a stop-valve to let out some portion of the contained air, if the pressure inside becomes too high. During operation the pressure inside is kept near about 100 lbs. per square inch.

The oil-pumps in the other two governors are of the gear-pump type. The pulley attached to the shaft of one of the gears is driven from the turbine shaft by means of a chain whose links are of reinforced rubber Double pulley and chain arrangement has been used. The other gear, of course, runs, being in mesh with the former. The gear-pump is constantly driven when it operates, and so there is a continual supply of oil under pressure, to the relay-valve cylinder direct from the pump, there being no oil reservoirs in these governors. The oil in its passage to the relay-valve cylinder first comes to a stop-valve, which is kept closed, when hand-controlling of the speed is resorted to. When oil-governing is going on, it is kept open. The bottom of the stop-valve is a little chamber with an escape-valve at the side and the oil goes to the relay valve through this chamber. The escape valve is ordinarily kept closed by a spring whose tension can be adjusted from outside. The tension of the spring is kept such that the oil has to attain a pressure of 60 lbs. per square inch in order to open the valve and get an escape. When no oil is being distributed to the main piston, *i.e.*, when the speed of the governor is not varying, the oil, continually supplied to the chamber from the gear-pump, pushes the escape valve open and gets a by-pass to the oil-basin at the base of the governor. This ensures also that whenever some oil is sent to the pressure cylinder it is sent with a pressure of 60 lbs. per square inch.

The regulation of hydro-electric units requires a certain departure from normal speed before the governor can act. Since the immediate effect of the gate motion is opposite to that intended, the speed will depart still further from the normal, which, in turn, tends to cause the governor to move the gate too far, with the result that the speed will not only return to normal

as soon as the inertia of the water and the rotating parts is overcome but may rush far beyond normal in the opposite direction. In other words, as the speed falls, the gates are opened and the supply column is accelerated --this action persisting until the supply of energy per unit time is equal to the demand. But the acceleration of the water-column goes on for an appreciable time, after the gate-opening has ceased and in consequence the supply becomes too great for the requirements of the wheel, the speed increases, and the governor commences to close the gates. This checks the motion of the supply column, and in virtue of its inertia produces an increased pressure at the valve and a temporarily increased velocity of flow through the gates. The speed of the wheel thus increases still further and the gates are closed until an instantaneous balance is again set up between the supply and the demand. As the inertia pressure falls, the supply now becomes less than the demand, the speed falls, the gates commence to reopen and the state of "hunting," which is here outlined, may not die out for some considerable time.

The governor, for successful operation, should bring the gates to their proper position with as little hunting or as few oscillations as possible about this position. To prevent hunting, it is necessary that the governor should cause the gates to overrun their final position slightly and to bring them back slowly to their final position corresponding to the altered load. This latter operation is performed by means of some form of 'compensating device.'

The 'compensating mechanism consists essentially of a dash-pot' or a cylinder in which works a piston connected with the main piston of the governor through levers. This dash-pot piston has got two holes in it, kept closed by spring-loaded valves. Nos. 3 and 4 governors have got their dash-pot pistons perforated with five holes. The cylinder is kept full of oil and the piston moves with some friction inside it because of the oil. In the case of Nos. 1-2 governors, the dash-pot cylinders have got racks attached to them which engages the pinion X shown in the sketch of the centrifugal governor. Because of the resistance of the piston inside the cylinder, the latter is bodily moved against the tension of a spring and the movement of the rack produces a rotation in the pinion X. This rotation of the pinion X either shortens or lengthens the rod RS in the governor, and thus adjusts the position of the relay-valve piston in such a way that the governor overshoots the mark. The tension of the spring then makes the cylinder come back to the normal position at a slow rate. This prevents any hunting that might take place

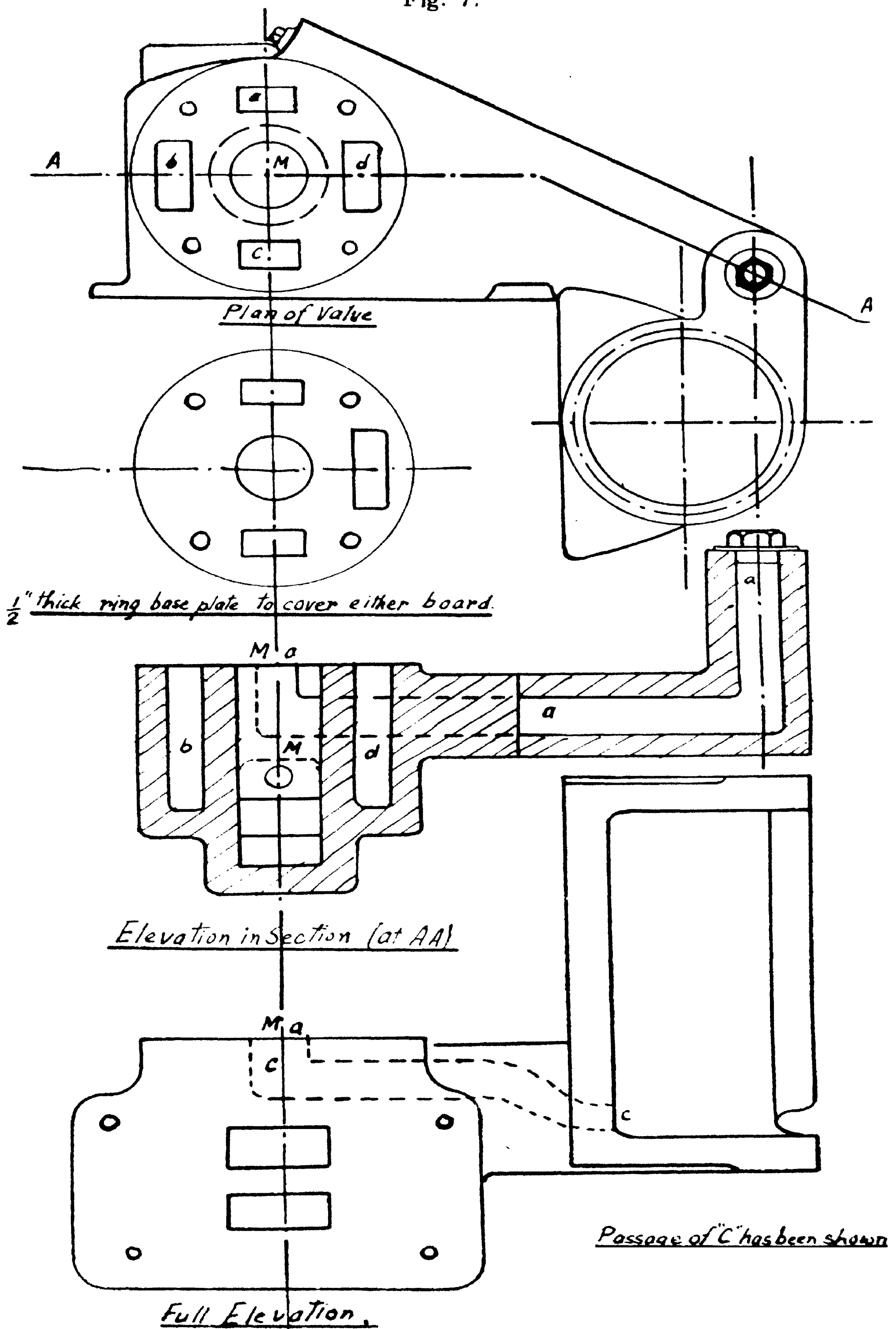
if the compensating mechanism was not present. In the 3rd and 4th governors the movement of the dash-pot cylinder up or down changes the position of the fulcrum of the top horizontal lever, to which are connected the centrifugal governor spindle and the relay-valve piston rod. The movement of the fulcrum also serves the same purpose of presenting hunting in the turbine, so that the gates move slightly beyond the final position, the final motion of the gates being a slow movement back to the final position.

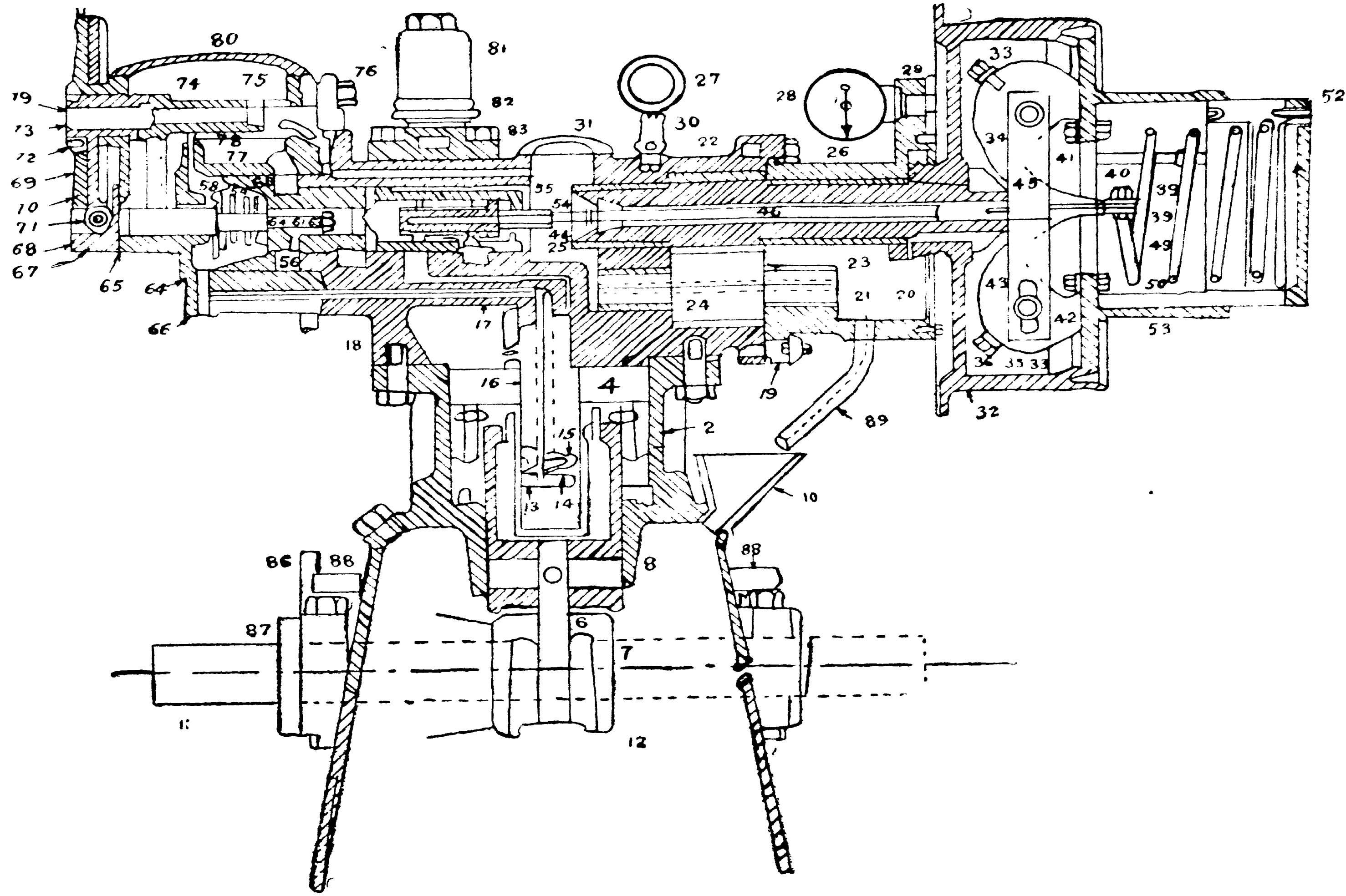
All the governors can be controlled by hand, *i.e.*, the pressure cylinder pistons can be moved by hand if the oil is stopped from doing its duty. In the first type of governors in the Jammu power station, the hand-regulating wheel has a pinion on its shaft which engages the rack fixed on the long piston rod which operates the gates through levers. In the other type, when the oil stop-valve is kept in the 'out' position, the oil does not reach the relay valve at all and then the speed of the governor can be varied by means of a hand control.

At the back of the piston there is a long screwed rod with square threads extending over a foot and a half from the end. This rod passes through a stuffing box on the cylinder cover so that no oil may leak out. The hand wheel runs freely over this threaded rod. Outside the stuffing box is a locking arrangement with two positions. When this locking arrangement is left in the 'oil-control' position, the piston can move either way along with the threaded rod at its back and the hand wheel on the rod. In the other position the piston is locked in such a way that it cannot move in or out unless the hand wheel is rotated by hand.

Boving Company's Governor at Jammu.—A photograph and a *sectional drawing* of this governor are shown. It consists of the following main parts:—(1) Governor body forming the oil tank, (2) servomotor, or power cylinder in which (3) a differential piston moves. The piston is connected through link (6) and lever quadrant (12) to regulating shaft (11), which, in turn, is connected with levers and rods to the turbine through regulating apparatus—gear wheel (22) pump which raises oil from the tank and pumps into the regulating valve. The pressure can be regulated by means of (81) the safety or overflow valves to correspond to the power required to regulate the turbine. After the valve has been adjusted, it should be permanently locked. Centrifugal pendulum (33, 34 and 35) of patent involute type is built into the belt pulley (32). It consists of two weights (33) fastened by two broad steel bends (34) at one end to the pendulum cover (53) and at the other to a cross-piece (39) to which the pendulum

Fig. 7.


A
M
b
d
c
A
Plan of Valve
1/2" thick ring base plate to cover either board.
M a
a
a
b
M
d
Elevation in Section (at AA)
M a
c
c
Full Elevation.
Passage of "C" has been shown



Boving & Co.'s Governor

To face page 504.

spring (49) and regulating pin (46) are attached. The links (41) serve a double purpose limiting the stroke of the pendulum and acting as a damper. The pendulum being quite free from friction, it would be too sensitive without these links so that the servomotor would respond to the smallest irregularities of the drive which would result in wear of the turbine-regulating apparatus. A continuous slight variation of the pendulum position, and hence the regulating pin, does no harm as long as it does not cause the servomotor piston to 'hunt.' Regulating valve consisting of a differential piston (54) in a movable liner (56) which is combined at one end with the piston of the compensator. The differential piston (54), the position of which is controlled by the pendulum through regulating pin (46), regulates the passage of oil to or from the upper side of the servomotor piston (3), the under-side being always connected to oil pressure. Compensator consisting of a cylinder (64) and piston (56) in one piece with the valve liner double acting spring (59), which always brings the piston back to the same position relative to the central spindle (65) and a sliding valve (61), which controls the passage of oil between both sides of the piston. The compensator is hydraulically coupled to the servomotor piston in such a way that when the servomotor piston moves upwards the compensator piston is pushed in the direction towards the pendulum; and when the servomotor piston moves downwards, the compensator piston is dragged in the direction away from the pendulum, the motion being in both the cases transmitted (by pressure and suction) by the oil between the compensator piston and the small cylinder of the fixed plunger (16). These movements of the compensator piston are always in the same direction as those of the regulating

Fig. 8



Boving Company's Governor
P. H. Jammu (Kashmir.)

valve piston (54) ; so that when the turbine speed drops, the regulating pin (46) and regulating valve (54) move towards the pendulum and open communication between the upper side of the servomotor piston (3), and discharge when this piston (3) moves upwards and presses compensator piston (56) and valve lining, so that it follows the valve where by communication with the discharge will be closed, the spring (59) then brings compensator piston slowly back to its middle position. The central spindle (65) is connected through levers (76 and 87) and connecting rod (86) to the regulating shaft ; so that the piston of the spindle will be dependent on the gate opening, and that a permanent speed drop between no-load and full-load of 2% to 3% is obtained, which is necessary to enable the machines to divide the load equally. Mechanical hand-regulating gear consists of a worm wheel and worm, the spindle of which is guided in an eccentric bearing so that it can be brought in or out of mesh by turning a handle. The bearing is at the same time the plug of a cock, through which the pressure side of the pump is connected to the oil tank, when the worm is in mesh so that the pump runs light. Cooling piping, for the circulation of the oil escaping from the overflow valve, through a pipe immersed in the turbine chamber, which will be full of water when the turbine is working and thus bring the cooling effect.

Troubles and Remedies :—Like all oil-pressure governors, this governor is also liable to troubles of which 'hunting' is to be treated with special care. The governor moves to and fro whilst the load is constant. This process is known as hunting. Before investigating the governor it should be ascertained whether there are any oscillations in the open flume which may be caused by the other turbines and which should result in oscillations of the governor. For this purpose the governor should be put on hand-regulating gear and the turbine speed observed by means of a hand tachometer to see whether there are variations in speed in spite of the load being steady. It is possible also to use for this purpose the tachometer mounted on the governor, but as it is affected by the irregularities of the bell drive, it is only dependable if the drive is quite correct.

(1) Ascertain by opening the air valve (78) in compensator, whether there is any air in the compensator. If this is the case, allow the air to escape and then close the valve. Observe after a few minutes whether any air has come in. If there is a leak, see whether the screw (66) and nut which fixes the central pin (65) (in compensator for constant steady speed) are properly packed. If these are air-tight, the leakage must occur in the joint between

compensator cylinder (64) and the body (18), which should be inspected.

(2) Close the turbine and ascertain whether the regulating valve piston (54) moves easily.

(3) Test whether the compensator is at rest in the middle position. In order to verify the middle position an indicator is provided by an extension of one of the screw, holding the cover (65), and which has a groove opposite which a pointer is fixed. After the turbine has been emptied of water, the gates should be opened quickly by means of the hand-gear. The compensator piston then follows the motion, goes afterwards back, first quickly until $\frac{1}{4}$ mm. from the middle position, afterwards it moves very slowly until the middle position is reached. For an open-flume turbine, when the oil has reached the normal working temperature, the 1st porti on of the stroke should be carried in 10 to 15 seconds and the last $\frac{1}{4}$ m m. should take as much time; when the opening and closing times are longer, the compensator time should be correspondingly longer but never more than $\frac{1}{2}$ a minute.

The turbine should be closed then quickly and the compensator time should be as above. The time can be adjusted by means of screws (62) and (63) in the compensator slide valve. The motion in the direction towards pendulum, which occurs after a movement of the governors in closing direction, is controlled by screw (62) in the groove farthest from pendulum. These times are, however, adjusted, and should not be altered without any good reasons. It may, however, occur that the time for the last $\frac{1}{4}$ mm. is too short and this may be due to the air valve (77) not being tight; in which case it should be inspected and put in order.

Make sure whether the leather washer (15) is tight. If it is not, the compensator piston may, for instance, not come back to its middle position when the oil pressure is on, but would stop about $\frac{1}{4}$ mm. from the middle position on pendulum side. The leakage may be due to the nut, holding the leather washer not being properly screwed down, or to the surface of leather being greased.

When assembling this detail care must be taken that the radial hole in the washer (14) should be opposite the corresponding hole through the thread; as otherwise the air accumulated at this point would not escape.

Check whether the pendulum speed drop (*i.e.*, difference in R. P. M. for the two-end positions of the pendulum pin divided

by the normal R. P. M.) corresponds to that stamped on pendulum casing. By increasing the speed drop, which may be obtained by using fewer turns of spring (49), the pendulum will become more steady, but at the same time the speed variation will be increased. See that the links (41) do not join against washers (42). It may occur that after sometime a ridge will form through wear, which may cause friction, and which should be removed.

The next trouble is the governor closing the turbine in spite of the speed being too low. This may happen when the governor is put in operation and the oil very cold and thick. After the governor has run for an hour and the oil has become warm, it will work satisfactorily. If, however, this fault should occur after the oil is warmed, it is caused by dirt entering the regulating valve and chocking one of the canals. The valve should be dismantled and cleaned Do not use a furry cloth when cleaning the valve and wash it afterwards with paraffin. In a governor with a long closing time the regulating valve is very sensitive to dirt as the openings are very small.

Still another trouble is the governor opening the turbine gates in spite of too high a speed, which is contrary to the previous one. It is caused by dirt in the regulating valve.

It will also be noted that the speed does not go back to normal after a change in load. This is caused by friction, wear, or possibly dirt in compensator, which causes the latter to stick. It may happen also through the threaded ring (57), forming seat for the compensator spring, being screwed in too much or too little. It should be screwed just so far, that no play can be noticed on pin (65) in axial direction.

The first of the possible troubles is the 'chattering' of the relief valve. This may occur when starting the governor with cold oil. The noise disappears if the pressure is reduced for a short while or if the hand-regulating gear is thrown on and then disconnected after a short while.

Maintenance and Inspection :—After the governor has been in operation for about two months, all oil should be emptied and the governor should be thoroughly cleaned and washed with paraffin, none of which should be allowed to remain in the oil tank. The governor should then be fitted with new oil or with old oil after careful filtering. Do not forget the sieve when filling. This complete dismantling and cleaning should afterwards take place once in a year, all parts being inspected and cleaned, the nuts, keys, etc., checked, and oil filtered. The

regulating valve itself should be dismantled and cleaned once in a month. As packing between the pump cover (19) and the body, only paper should be used of the same thickness as put in at the works. The paper should be first greased to ensure a tight joint.

When dismantling the governor, attention should be paid to the marks indicating how the parts fit together, and in case the adjustment is altered new marks should be made and a record kept so that the parts may be re-erected correctly. The dismantling and assembling of pendulum should be effected in the following manner. First slacken all the screws holding cover (53) against pulley (52) and remove carefully the cover with pendulum and pin. In case that there is not sufficient space to take it right out, the pin can be disconnected by taking off the split pins (47). Then screw back the screw (89) in cover (53), remove cover (52) and unscrew the adjoining screws (50) until the pendulum spring (49) is completely slack ; after which it can be disconnected from the pendulum cross-hand and removed with its nut. Then remove the stoppings (42) and links (41) ; loosen set screws (36) and disconnect weights (33) from the steel band. When assembling operate in reverse order taking care after the spring has compressed a little that each steel tape lies properly against its weight by shifting the latter slightly. For the lubrication of the pendulum and steel tape, only oil or grease free from any trace of acid should be used, otherwise the steel tape will rust.

Governors, in Mahora Generating Station, and Method of Governing :—For the speed regulation of impulse wheels, there are four methods in general use :—

(1) By deflecting the jet from the wheels by means of a deflector placed in front of the nozzle tip.

(2) By needle regulator, regulating the size of the jet.

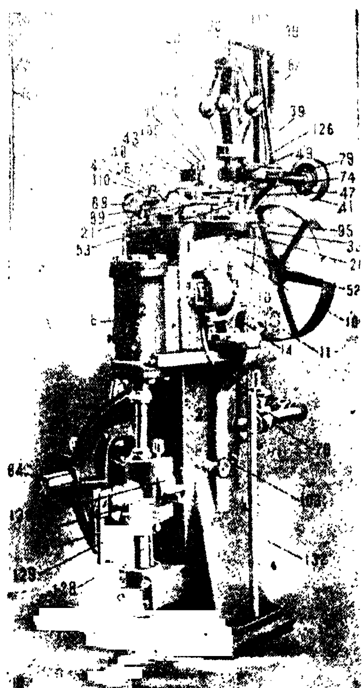
(3) By combined needle regulator and deflecting jet.

(4) By using a diffuser.

In the first method the jet is deflected wholly or partially from the buckets of the wheel by the deflector, actuated by the piston working in the main cylinder of the governor, the motion being communicated by links to the shaft of the deflector on which the deflector is rigidly fixed and moves in an angular direction with the shaft. This method is rather a waste under a variable load. The loss may be reduced by using the deflector in conjunction with a hand-regulating needle. This is set by hand at intervals to give the maximum discharge likely to be required

during the next period and any fluctuation of load up to this maximum is handled by the deflector. This is the method employed for the four Pelton wheels in Mahora.

Fig. 9



Lombard Governor, Mahora.

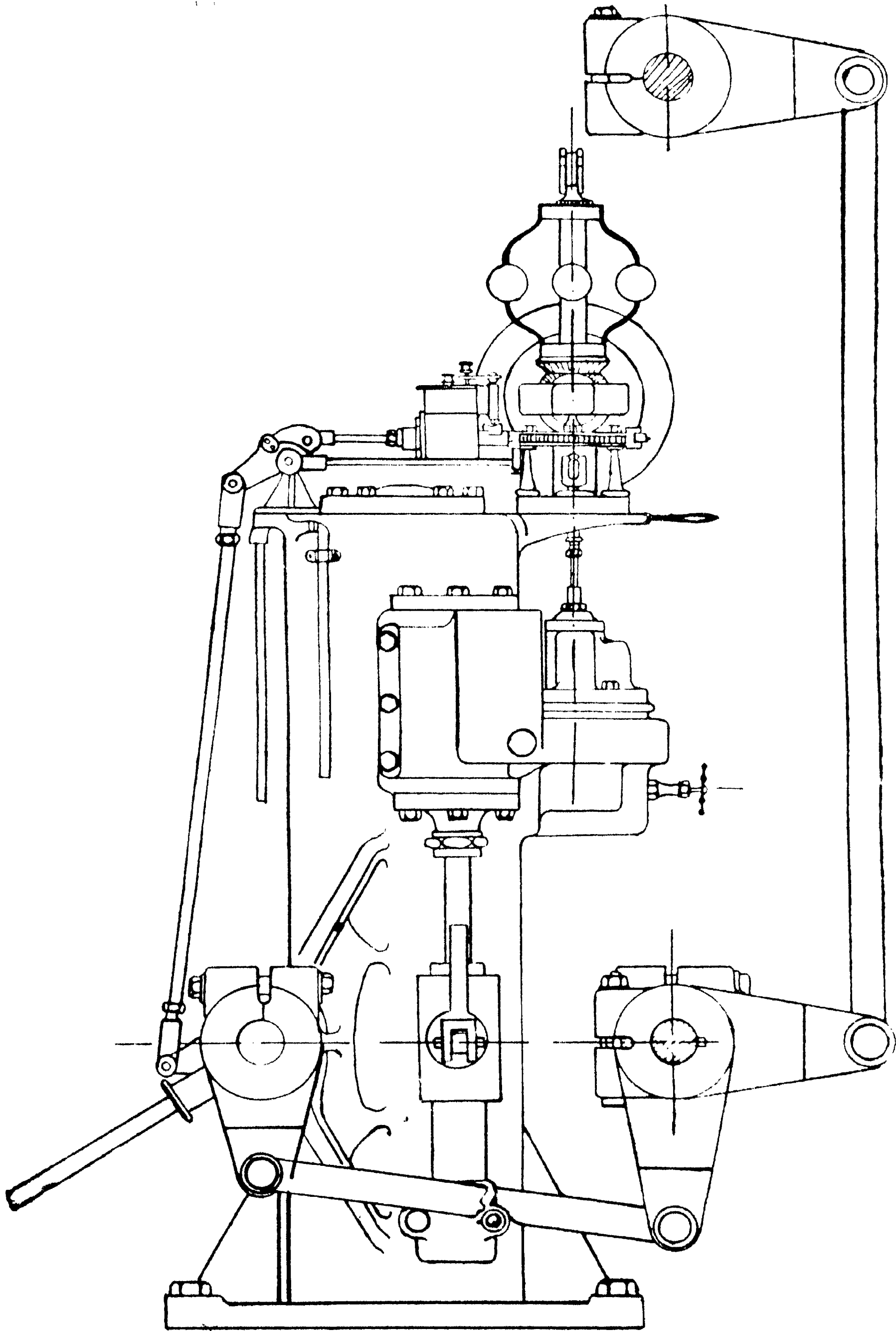
The second method of regulation is that the position of the needle is regulated by the governor. The piston, working in the hydraulic cylinder of the governor, is connected by means of links to the needle valve, which thus responds to the movements of the piston, caused by change in loads. The motion of the needle valve increases or decreases the size of the jet as the case may be. This method of governing is used for the two Pelton wheels driving the exciters at Mahora.

In the diffuser method of governing, which is of recent date and designed by the English Electric Co., there is a spider which is capable of sliding over the needle, and which is in flush with the needle valve surface under normal conditions of load When the machine races, the diffuser travels forward and sprays the jet, bringing about the speed regulation.

As mentioned before, all the turbines in Mahora are of the impulse type and each of these Pelton wheels is supplied with the **Lombard Co.'s** oil pressure governor. The governors are exactly the same as the one in the *Jammu power-house* with the exception that in these the oil tanks are vertical ; whereas in the one at Jammu, the oil tank is horizontal. To the piston, working the hydraulic cylinder, is connected a load placed in front of the nozzle by levers, so that when the load is deflected the jet is diverted. The needle valves are controlled by hand.

The governors for the two Pelton wheels driving the exciters, are also of Lombard type, but these two governors have only one common oil tank, which, like other governors, has two compartments—the pressure tank and the vacuum tank. The oil is pumped by means of a reciprocating pump driven by a direct-current motor. In this governor, the piston rod is connected to a sliding block which moves to and fro in a V-groove. The rod connecting this slide to that coming from the

Fig. 10



Lombard Governor back-view.

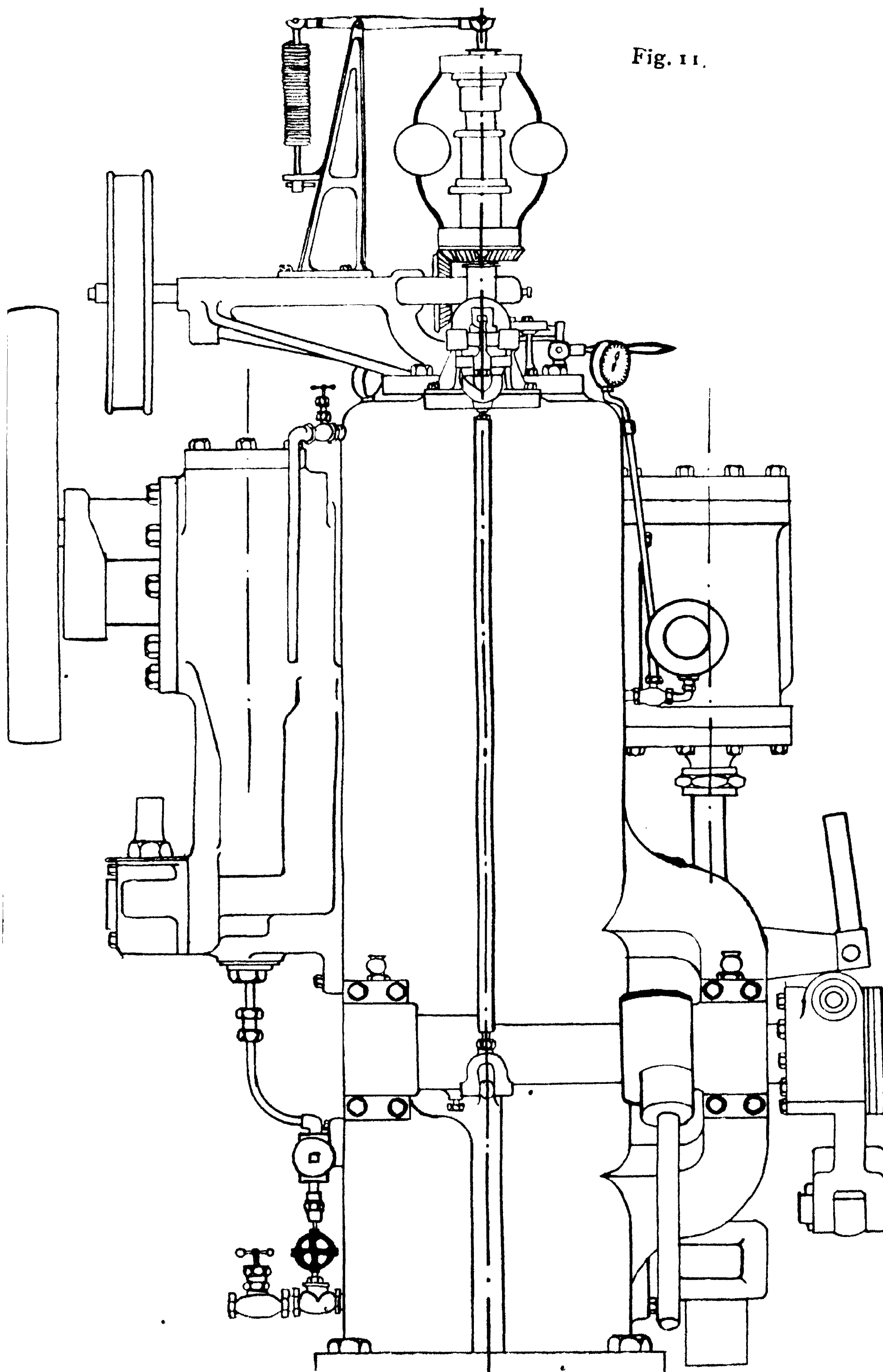


Fig. 11.

needle valve is in three parts. The central one is hollow and has threads cut in it so that the two rods on either end of it can be screwed in or out, as the need may be, so as to adjust its effective length Through these arrangements the needle valve receives motion from the main piston.

The main difference of this governor from that of Boving's is the method of pumping employed. The pump used in this governor is a reciprocating one. This pump pumps oil from the low pressure, or vacuum tank, as it is usually called, to the high pressure tank, which supplies oil under pressure to the working cylinder, so as to open or close the turbine gates, as the case may be. This pump is driven by belts connected to the turbine shaft. Another difference is in the construction of the centrifugal head which is exposed in this case. The compensator also is much different from that of Boving's governor. The troubles of this governor are also the same and the remedies, care and maintenance referred to above apply here, too, as well.

Lombard Governors :—The governor consists of the following principal groups of mechanism :—

(1) A combined pressure tank and receiver with a complete piping system connected thereto.
(2) A power pump driven by pulley.
(3) A hydraulic cylinder.
(4) A balanced regulating valve.
(5) A centrifugal governor head connected with regulating valve.
(6) Anti-racing mechanism consisting of lever, dash-pot, valve stem rack and pinion.
(7) The terminal connections consisting of main rack, pinion, clutch and hand-wheel shaft.

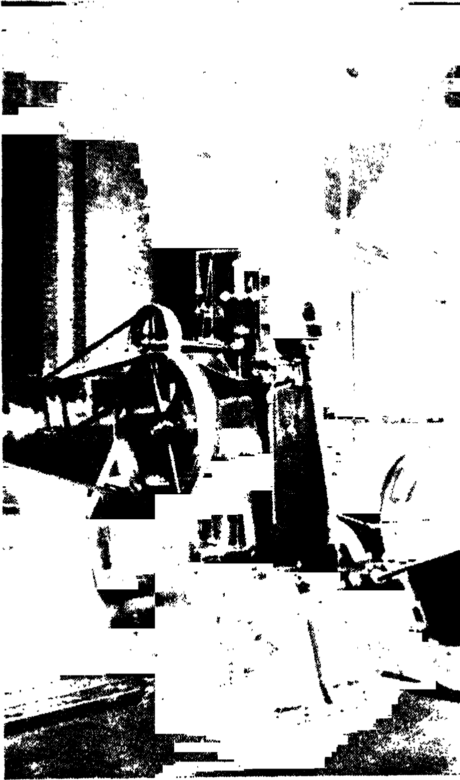
The governing in this case is effected by a hood which changes the angle of the jet, when it goes to meet the Pelton wheel. This hood is automatically worked by the governor.

The pressure tank containing air and oil, in nearly equal volumes under a pressure of from 100 to 200 lbs. per sq. in., serves as a source of energy for the hydraulic cylinder of the governor. The smaller compartment serves as a receiver of oil exhausted from the hydraulic cylinder.

The function of the pump is to remove the oil from the receiver as fast as it accumulates and return it to the pressure tank. But it is observed that the pump restores this energy at a constant and comparatively slow rate, while the hydraulic cylinder sometimes uses this energy from the tank very rapidly. This fact is

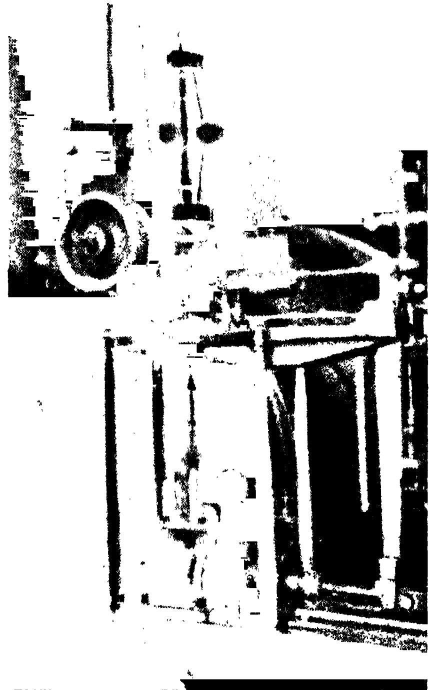
of utmost importance in speed regulation, for it is important that the turbine hood should be moved as quickly as possible,

Fig. 12.



Lombard governor at Mahora P. H. as in connection with Pelton wheel

Fig. 13.



Lombard governor of the exciter panel

when they are to be moved at all; because during all the time, the hoods are being moved, the speed of the turbine continues to deviate from normal. The pressure tank is analogous to storage battery, for it receives energy gradually and gives it out occasionally, but at a very rapid rate.

It is obvious that the energy in the pressure tank is contained in the compressed air, filling the upper half of the tank, which pushes continuously on the surface of the oil beneath.

The oil from the pressure tank is supplied to the working cylinder, through the regulating valve arranged to discharge or exhaust oil directly and very rapidly, into or from either end of the cylinder.

The connection between the centrifugal balls and the regulating valve is by a valve stem in two parts, connected by a screwed coupling, so arranged that the total length can be increased or diminished. The normal running speed of the governor may thereby be altered. The valve stem terminates above, inside the top of the governor head, in a very steep pitch screw. This screw may be turned in a nut, which cannot rotate but which moves up and down with the flange, to which the upper ends of the flat springs carrying the balls are fastened.

The portion of the valve stem, above the adjusting coupling, carries a pinion, which engages with the rack, normally held in a central position by means of tension springs. The part of the valve stem, containing the steep-pitch, screw-pinion rack, is a portion of the antiracing mechanism.

Through the lever, connected in a simple manner to the main rack, reduced motion is transmitted to the dash-pot rod, and thence to the dash-pot itself to the spring centred rack.

The Antiracing Mechanism.—Almost always, when the governor becomes out of order, the speed varies periodically up and down from the normal, and it is called to race.

The chief causes of racing are :—

(1) It is operated at too low pressure;
(2) The pump being out of order;
(3) The regulating valve is worn or leaky;
(4) The turbine blades bent;
(5) The driving pulleys being eccentric;
(6) The governor oil is old or dirty and gummy;
(7) If there is air in dash-pot;
(8) If the antiracing mechanism does not work properly;
(9) If there is lost motion in the connections between the governor and turbine blades;
(10) In some cases it might occur on account of badly-arranged draught tube.

The antiracing mechanism is an essential part of the governor, which is demonstrated as follows :—

Assuming that it does not exist, and that there is a plain connection between the governor top and the regulating valve, that is the valve stem is of unchanging length, then let a small change occur. Instantly the centrifugal balls will move to throw the piston rod out or in. This movement would continue as long as the speed is away from normal. Hence, the hood will completely open or close the jet long before the speed has come

back to normal. As a result, the speed will not come to normal but will go beyond in the other direction, reversing the displacement of the valve causing the piston to travel in opposite direction. This will be indefinitely repeated and is known as the *racing of the governor.*

The actual condition of affairs, however, is such that the dash-pot forms only a temporary connection from the upper end of the lever and neither lengthens nor shortens the valve stem permanently.

If the opening of the adjustable valve is slight, the dash-pot will give way to the spring very slowly ; if the opening is great, rapidly.

Bringing the turbine up to speed under the control of the governor :—The clutch pin is first thrown out and the hand wheel pinion thrown in, so as to remove the control of the turbine from the governor. Then the gates are opened by hand, until the speed is normal. After that, the throttle valve of the governor is opened wide; when it will be found that the governor piston moves in one direction or the other, it is brought to the center of its stroke by touching the hand lever. If the piston moves at all rapidly in either direction, turn the coupling so that the approximate equilibrium is reached. Then by tightly touching the handlever bring the piston of the governor to the same percentage of stroke as the hood is of its full travel. Then the hole in the clutch disc will register with the clutch pin and should be thrown immediately. The hand wheel pinion should be disengaged and fastened out to the mesh.

If this is not attended, the pinion goes off the inertia of the handwheel. The safety lever is then lifted so that the clutch is released and it is held up by the trigger.

Care and Maintenance.—The safety lever is liable to be left fastened up after starting. The adjusting coupling sometimes becomes loose if the check-nut above has not been fully fastened. The regulating valve sometimes sticks because of the decomposition of the oil. or foreign matter which passes the filter. Observe that the stem is constantly vibrating through a small range. Touch the hand lever and note whether it returns immediately to its regular position. The pressure should be nearly normal except for brief fluctuations. The safety valve should not blow off much air. Pump-bearings should not be much heated. The sound of the pump piston should be regular and well-defined. *At occasional intervals*

the screen from the filter should be cleaned. Oil level in pressure tank should not be low. Oil from the drain cock should be allowed to stand in a clean test-tube and the sediment should be estimated.

Every time before starting, it should be seen that the belts are sound. The connections to the blades are examined, and it is seen that there is no lost motion or worn part. All the bearings are lubricated with good grade lubricating oil. If the oil-pressure gauge does not indicate the pressure high enough, the governor clutch must be disconnected and the gates opened by hand until the pump has raised the pressure to the necessary amount. Dash-pot chamber should be full of oil to its working level.

Automatic Oil-Pressure Governors (Shivasamudram)

(Type U.K. 2, 3, 4 & 5 Instructions from Makers.)

The governor consists of the following main parts. See figure attached.

(*i*) Governor body, of which the lower part (1) is the pressure chamber. The upper part (2) is the oil reservoir and also contains the servomotor (3).

(*ii*) The gear wheel pump (8-16), which draws the oil from the reservoir (2) through pipe (14) and pumps it into the pressure chamber through pipe (99), check valve (15) and delivery pipe (16).

(*iii*) Centrifugal pendulum (17-34) of the evolvent design (17) is raised or lowered by changes in speed and transmits the movement of the pendulum weights to the lever system.

(*iv*) Lever system consisting of lever (84) and valve rod (85), which transmits the movement of the pendulum to the pilot valve (38) which controls the regulating valve (40).

(*v*) The regulating valve consists of valve (40) in a liner (41). The valve (40) follows the movement of the pilot valve (38); so that when it moves upwards, pressure oil is admitted to upper end of the servomotor piston (4), which is forced downwards. When valve (40) moves downwards, the oil escapes from the upper end of the servomotor piston and the latter is forced upwards by the oil-pressure in the annular space on the lower side of the piston (4). This space is always connected to the pressure chamber (1) when the stop valve (66) is open, *i.e.*, when the governor is on automatic control and the hand control is out of gear.

(*vi*) The servomotor or power cylinder (3) in which a differential piston (4) moves. The piston is connected through link (5) to the lever quadrant (6) connected to governor shaft (7), which, in its turn, is coupled by means of levers and links to the turbine guide vane-apparatus or to the Pelton needle valve gear.

(*vii*) The return gear, which is a connection between the governor shaft and lever (84) acting so as to return the pilot valve (38) and consequently the valve (40) to normal position after a movement, has been caused by the pendulum acting on a change of speed.

(*viii*) The compensator is a part of the return gear and consists of a sleeve (47), a container (46) and a piston (45) with a connecting sleeve (48) and a compensator valve (49). The piston (45) is kept in a fixed position relatively to the cylinder (46) by means of washers (42, 44), but moves from this position when the container (46) rises or falls. The compensator valve (49) is rigidly fixed to the sleeve (47). When the container (46) rises or falls, the piston (45) follows it, but afterwards returns slowly back to its normal position. The first portion of this return should take 12 to 14 seconds with turbine erected in open flumes and the last half mm. should take equally long time. This slow return is affected by the spring (43) after an upward movement. The flow of the oil between the upper and lower ends of the piston is regulated by grooves cut in the valve (49). These grooves are closed when the piston is in the normal position, but are increased as soon as the piston (45) moves upwards or downwards.

(*ix*) The minor eccentric spindle (56) is coupled to the sleeve (47) by means of link (59) when running in parallel with another set; the eccentric (56) is adjusted so, that the movement of sleeve (47) is about $1\frac{1}{2}$ to 2 mm. for the whole compensator and in the same direction as the movement of sleeve (43). This adjustment is affected by turning the square-ended spindle (56), and can be checked in the same way as the return movements of the compensator is, by raising and lowering the return lever (78), through an angle equal to that, which the governor shaft moves when properly connected up. The speed, altering the height of the lever (84), consequently changes relative heights of (17) and pilot valve (38).

(*x*) The alteration in speed may also be effected by a small motor (57) with suitable gearing (86) extra. The mechanical hand-regulating gear consists of hand valve (60), which is connected

to the regulating shaft (7), through the gear wheels (61, 62 & 63) and the segment (6). The shaft carrying the worm (61) is placed in an eccentric bearing (64), which can be turned by the hand lever (65). By means of the worm, (61) may be brought in or out of mesh with the gear valve (62), and the hand control coupled in or out, as desired.

(*xi*) The stop valve (66) is placed between the pressure vessel and the control valve body (39). The stop valve is connected to the hand lever (65), by link (67), so that when the hand-control gear is coupled in, the pressure oil is cut off from the automatic control valve and vice-versa. In this, above, is a special oil passage, which allows the oil to escape from the servomotor, when the hand gear is in service.

(*xii*) The safety valve (35), placed on the pressure vessel (1), prevents the oil pressure from increasing above a pre-determined value, which can be regulated by the adjusting screw (36).

When an unloading valve is supplied with the governor, the safety valve is set to open when the pressure has increased about 1 Kg. per sq. cm. (14 lbs. per sq. in.) over the maximum pressure at which the unloading valve will remain open. If the unloading valve filter becomes clogged, or the connection (96) becomes blocked, the safety valve will then function and thus prevent an excessive pressure in C.

(*xiii*) The unloading valve (92-93) is connected to the oil-pressure pipe, between the gear-wheel pumps and the non-return valve (15), and to the pressure vessel by means of a pipe through the connection (97) valve (96) and the filter (95). The unloading valve regulates the pressure in the air vessel, by automatically opening and admitting oil under pressure from the pump to the pressure vessel, when the pressure sinks below a fixed point, and by-passing the oil from the pump into the reservoir (2), when the pressure in (1) has reached the maximum figure for adjustment (93 and 94). The unloading valve is only supplied extra.

(*xiv*) The outlet valve (71) connects the pressure vessel to the air reservoir. This valve is for adjusting the level in pressure vessel and is hand-operated.

(*xv*) The indicator (75) is graduated to show the amount of guide vane opening on the turbine.

(*xvi*) Cooling pipe (73) is provided when the governor must work with high oil-pressure. Cooling is provided which may be arranged either for a circulation of water through a coil in the oil tank (2) or for the circulation of oil escaping from the safety

valve through a pipe immersed in the turbine chamber or discharge canal back into the oil tank.

(*xvii*) The tachometer (109) is graduated to read in periods or in R.P.M. of the turbine shaft. Extra ordered and specified.

Governor of Mandi (Vide Fig. 16, Turbines)

1. **Governor Function** :—The governing of each water-wheel is effected by a sensitive pendulum controlled by means of oil under pressure through regulating valves and servomotors, the movement of a needle limiting the jet from the nozzle and the movement of a deflector plate acting on the jet. *Vide* pp. 471-475.

All necessary gear has been provided to meet the following requirements :—

(*a*) To ensure, under normal conditions, full compliance with the requirements of clauses 7-9 (page 513) inclusive and also entire freedom from any tendency to hunt or race.

(*b*) The remote control of speed from the switchboard for synchronising and load adjustment under normal conditions.

(*c*) The hand control of the deflector and needle under all conditions of working, including the conditions obtaining when the pendulum operating valves, compensating device and integral oil pump of the governor are dismantled whilst the machine is on load.

(*d*) The automatic limitation of the water-wheel output to a maximum of 17,000 B. H. P. under any particular effective head.

(*e*) That when normal oil-pressure is available, any movement of the deflector, automatic or otherwise, is followed by corresponding movements of the needle at the rates described under "*Closing and Opening Times.*"

(*f*) When the deflector and needle are under normal pendulum control (*i.e.*, when oil-pressure available) and oil-pressure fails, each is automatically and independently locked in the position it occupied at the time of failure of oil-pressure, and thereafter it is controlled by hand at the machine excepting in the emergencies described in sub-paragraphs (*i*) and (*j*).

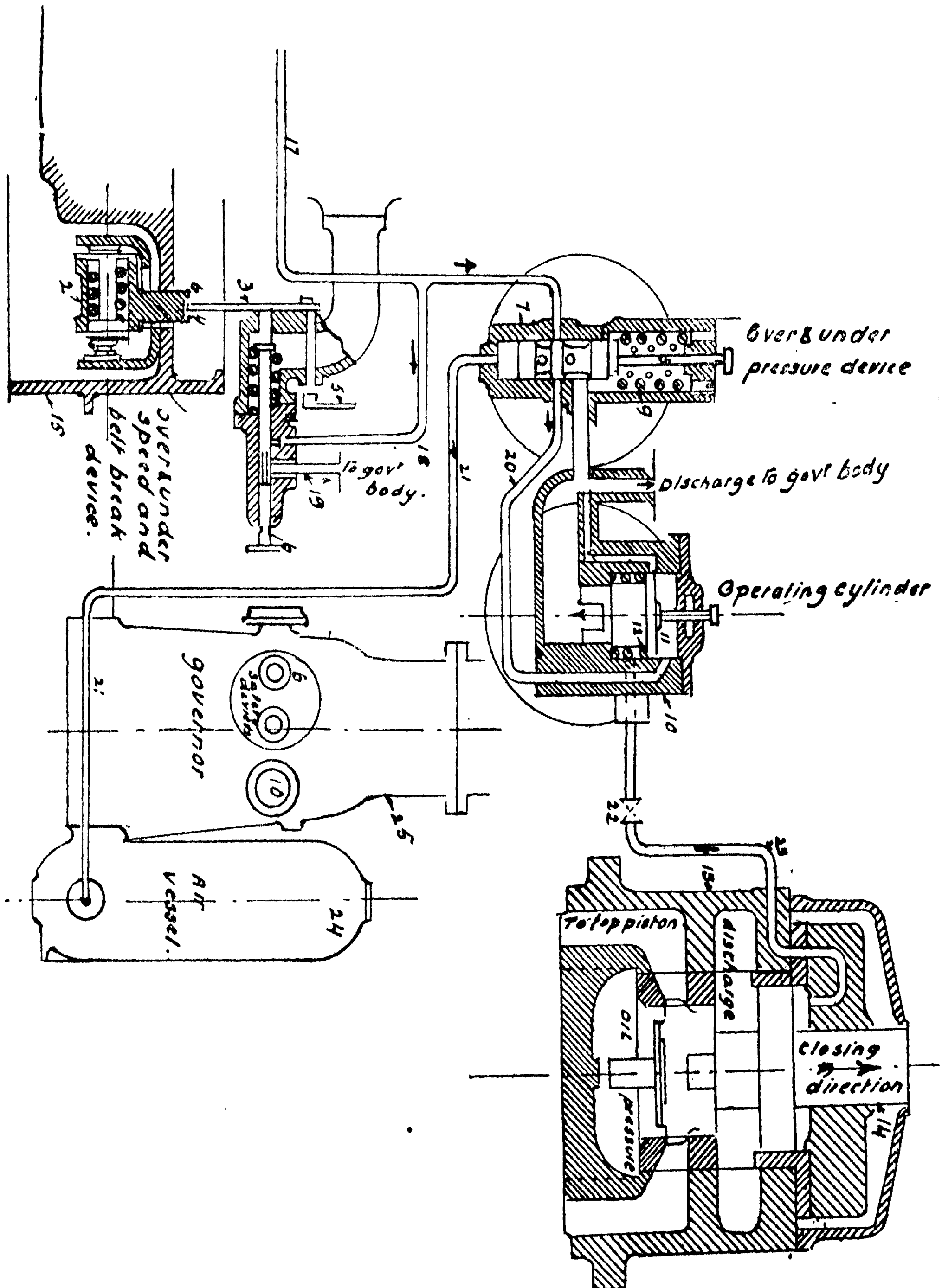


Diagram of Safety Devices for Governor V. K. H. for Turbine No. 11 and 12.

List of parts—

(1) Oil pump for safety device.
(2) Under- and over-speed device.
(3) Trip.
(4) Operating valve for No. 2.
(5) Hand resetting lever.
(6) Piston.
(7) Safety device against A. B. normal oil-pressure
(8) Piston.
(9) Spring
(10) Operating cylinder for all safety devices.
(11) Piston.
(12) Spring.
(13) Regulating valve for governor.
(14) Piston valve.
(15) Belt pulley for governor.
(16) Suction pipe for No. 1.
(17) Pressure pipe for No. 1.
(18) Pressure pipe to 4.
(19) Discharge pipe from 4.
(20) Oil supply pipe to 10.
(21) Oil-pressure pipe.
(22) Cut-off valve.
(23) Release pipe.
(24) Air vessel.
(25) Governor body.

(*g*) When the deflector and needle are under normal pendulum control (*i.e.*, when oil-pressure available) and overspeed occurs, the deflector is automatically closed by oil-pressure at the rate of 0·8 second for the full stroke, and the needle is automatically closed at the rate of 8 to 30 *seconds*.

(*h*) When the deflector is under hand-mechanical control and oil-pressure fails, the needle is automatically locked and thereafter controlled by hand at the machine.

(*i*) When the deflector is under hand-mechanical control, whether the needle is under normal oil-pressure control or not, and overspeed occurs, the deflector is automatically closed by an electric motor, at the rate of 5 seconds for the full stroke ; and this closing is independent of the *emergency remote control* from the switchboard described below in (*j*).

(*j*) When the deflector is under hand-mechanical control, whether the needle is under normal oil-pressure control or not, and an emergency other than the overspeed occurs, the deflector is capable of being closed by means of the electric motor, described in the previous sub-paragraph, at the rate of 5 seconds for the full stroke.

2. Governor Mechanism :—

(*a*) The governor of each water-wheel is of standard design *manufactured by Verkstaden Kristenehamn of Sweden*, comprising a pendulum, the shaft of which is extended to form a pin valve rotating and sliding in a relay sleeve valve, which is moved hydraulically in an outer compensating sleeve, governor servomotor, air vessel, oil pump, main rocking shaft, return gear and hand-operating gear. The pendulum is of the type in which all sliding and rotational friction is eliminated by having the weights rolling on bands and the bands rolling on the pendulum head. The pendulum retains even sensitiveness over its entire stroke, and is driven from the generator shaft, by a specially wide belt of laminated leather of the *Hendry Laminated Type*. A suitable belt guard is also provided.

(*b*) The *servomotor* is incorporated in the governor assembly and the movement of its piston under oil-pressure is controlled by the pendulum acting through the *pin valve* and *relay sleeve valve.* The servomotor is normally supplied by oil from the governor air-vessel, which, in turn, is normally kept charged by the independent *oil pumping set* described in clause 5. The servomotor is of the differential type to ensure quick response and is of ample size for the complete control of the deflector under all conditions of nozzle opening.

(*c*) *The oil pump* is also incorporated in the governor assembly and driven by the same shaft as the pendulum. As oil is supplied to the governor air-vessel from an independent oil pumping set (clause 5), the governor oil pump is normally used to lift oil from the governor oil sump and circulate it under low pressure to keep the governor head lubricated. The capacity of the pump, however, is such that under extreme emergency conditions, when all other supplies of pressure oil have failed, the closing of a pump release cock, normally open, causes the pump to deliver a supply of oil adequate for the slow operation of the deflector and needle.

(*d*) The governor is not called upon to supply any power for operating the needle which has its own servomotor, operating valve and return gear (see clause 3).

(*e*) Each governor is provided with a speed altering device, operated both by hand and alternatively by a 220-volt direct-current motor, to enable the speed of the set to be varied as required from the floor or from the switchboard for synchronising purposes, or for adjustment of load between the sets. This motor is also used for shutting the water-wheel down, from the switchboard, if required.

(*f*) In order to maintain automatically an even balance of load between the generating sets, when the total load on the station varies, each governor is fitted with an automatic device to give the range of speed variation between no-load and full-load. This device is capable of being adjusted by hand whilst the machine is running.

(*g*) Each governor is fitted with mechanical control gear, which, when put into use by the operation of the hand-wheel on the clutch control valve, described below, automatically cuts out automatic control of the deflector and needle by the pendulum. In such case, when the oil pressure system is fully charged, the control of the needle continues by oil pressure, in the manner described in clause 3, as its operating valve responds to any movement made by the governor mechanical control gear, when used to control the deflector.

On failure of oil pressure, a toothed-wheel clutch in the governor normally kept disengaged by oil pressure acting against a spring, is automatically engaged and put the deflector on mechanical control. This clutch action is governed by a positive control valve of the trigger type, which ensures that the clutch engages fully and not partially. Normally, this clutch control valve is operated with pressure oil through a filter from the governor air vessel, but for the purpose described above it is fitted with a operating hand-wheel. Also on failure of oil pressure a locking valve, situated between the needle control valve and its servomotor, (see clause 3), closes and locks the needle hydraulically in the position it occupied when the oil pressure failed.

The mechanical control gear of the governor is equipped with a hand-wheel, and in order that the deflector in the cases of emergency described in clause (15), paragraphs (*i*) and (*j*), may be completely closed in 5 seconds ; it is also equipped with a chain drive from a 3 B. H. P. 220-volt direct-current motor. It is not required that load adjustments on the set be made by means of this motor.

(*h*) Each governor is fitted with a load-limiting device capable of being set by hand to restrict the output of the respective water-wheel to a maximum of 17,000 B. H. P. The device consists of a hand-operated variable stop, which, when set, prevents the governor operating valve from opening the needle beyond the desired gate opening. It does not interfere in any way with the normal governing below the maximum limit of output.

(*i*) The governor head, *i.e.*, the part of the governor which contains the pendulum, operating valves, compensating device and oil pump, is so constructed that it may be dismantled for repairs or inspection without having to stop the water-wheels. Whilst the

governor is partially dismantled in this way, the deflector is controlled by the hand-control gear; and as long as oil pressure is available, the movements of the needle automatically follow the movements of the deflector. Under such conditions the governor air-vessel is isolated from the oil system by closing a valve provided for this purpose at the point where the branch to the governor air-vessel leaves the pressure pipe from the independent oil-pumping set.

(*j*) An alarm device of approved design and arrangement is provided to indicate when the oil pressure fails.

3. **The Deflector and Regulating Needle Mechanism** :—

(*a*) The water-regulating mechanism of each water-wheel comprises a deflector, with operating gear, regulating needle, needle spindle, needle servomotor valve, return gear, locking valve, hand pump, position indicator and water by-passing arrangement.

(*b*) The deflector plate is of stainless steel and is operated mechanically through rods and levers by the governor servomotor piston. The main connecting rod is fitted with right and left-hand screw-threads, so that the relative positions of the deflector and regulating needle may be adjusted.

(*c*) The regulating needle is made in two parts, the main body of cast stainless steel and the tip of forged stainless steel of the same grade as is used for the nozzle tip.

(*d*) This needle spindle is made in two parts to facilitate easy dismantling. That part of the spindle which is outside the inlet bend is of mild steel, and that part inside the inlet bend is of stainless steel of such a grade as to obviate contact corrosion within the bronze bushings, through which it operates, and the necessity for any protective sleeves. A galvanised mild steel tube is provided in the inlet bent to protect the needle spindle from the abrasive action of foreign bodies carried through by the water. The spindle is equipped with a leather packing to ensure water-tightness.

(*e*) The needle is directly operated by a servomotor on the needle spindle, the piston of which moves in the opening direction under oil pressure acting against

a spring, and in the closing direction under the action of the spring as controlled by the rate of oil discharge. The admission and discharge of oil to and from the servomotor are controlled by a regulating valve, operated by a floating lever, which is coupled at one end by a rod to the deflector control shaft, and at the other end by a short link to the needle spindle, the latter connection constituting the return gear of the needle.

The ports, admitting oil and to discharge oil from the needle servomotor, are made so, that can be finally adjusted at site to suit operating conditions. The adjustment of the ports is made accessible to interference whilst the plant is running. When despatched, the timing ports had openings corresponding to a needle closing time of 60 seconds, and a needle opening time of 36 seconds. The timing ports are fitted close to the servomotor casing to ensure safety to the pipe line under any accident that might happen to the control mechanism.

(*f*) Oil pressure for the needle operating valve is supplied from the same source as that for the governor, *i.e.*, from an independent oil pumping set, and as speed of movement of the needle is to be controlled only by the sizes of the two timing ports, it is, therefore, not effected by the speed and amount of travel of the needle-operating valve.

(*g*) As long as oil pressure is available, control of the needle continues in the normal way, its operating valve responding to any movement made by the deflector, whether the latter is on pendulum or hand-control.

(*h*) In the event of failure of oil pressure in the system, a locking valve situated between the needle control valve and the servomotor timing ports closes, and locks the needle hydraulically in the position it occupied when the oil pressure failed. The needle is then opened and supplied by a hand oil pump provided for the purpose and connected between the locking valve and the servomotor timing ports. This hand pump is provided with isolating cocks, which normally are closed, and with a small by-pass pipe and valve for use in closing the needle when required.

On the re-establishment of oil pressure in the supply system the locking valve reopens automatically.

(*i*) The operating handle of the hand pump is removable so as to render the pump less liable to unauthorised interference.

(*j*) A hand-wheel capable of being locked in any position is provided for varying the length of the link on the needle return gear so that it may be possible to by-pass surplus water to any required extent up to full jet depending upon the load which the machine is carrying at the time.

This provision does not interfere with the operation of any emergency controls or with normal governing underload variations up to the load corresponding to full jet discharge less the amount of water by-passed.

(*k*) A special indicator is provided and fixed in an approved position to show the true relative position of the deflector and needle under any conditions of operations.

4. Emergency Governors :—

(*a*) An adjustable mechanical overspeed device of the centrifugal type is fitted to each generator shaft and is so set that at 15% overspeed a trigger is automatically tripped and thereby opens the contacts of the overspeed switches.

The overspeed device on the generator shaft, complete with trigger and overspeed switches, and the governor solenoid, complete with plunger, is provided in addition to the governor mechanism already described.

(*b*) The contacts of the overspeed switches are designed to open circuit on overspeed, but otherwise to remain closed, and to maintain the governor solenoid, and the operating coils of the pilot contactors permanently energised whilst the machine is running.

(*c*) In the event of overspeed the opening of the contacts of the overspeed switches results in the following :—

(1) When the water-wheel is running under the normal pendulum control (*i.e.*, with oil pressure available), the governor solenoid is de-energised and its plunger forces the governor operating sleeve valve into its closing position, thereby closing the deflector almost instantly (*0·8 seconds. See clause 1-g*).

(2) The deflector pilot contactor operating coil is de-energised allowing the respective "*closing*" pilot contactor to

close and energise the main contactor coil which closes the main "*closing*" contactors and thereby start up the emergency control motor described in clause 2 (*g*).

(3) Whether the governor is in commission or not, the valve pilot contactor operating coil is de-energised allowing the respective closing pilot contactor to close and energise the *main contactor* coil, which closes the main "*closing contactors*." The main inlet valve then begins to close and continues to close as its predetermined rate, irrespective of any attempt at remote control from the switchboard, until the contacts of the *limit switch* are mechanically opened on completion of the movement of closure.

(4) The de-energising of the deflector pilot contactor coil allows the "*alarm*" pilot contactor to close and thereby complete the circuit through the alarm bell and red-indicating lamp.

5. Oil Pumps and Oil Pressure System :—(*Important*).

(*a*) The pressure oil for the governor and regulating needle servomotors of each water-wheel is supplied from an *independent motor-driven oil-pumping set*. The pressure in each system is normally maintained at about 150 lbs. per sq. in. and does not exceed a pressure of 212 lbs. per sq. inch.

(*b*) Normally each oil-pumping set is to supply oil only to its respective governor and remain isolated from the inter-connecting piping system described below. The capacity of each pumping set, however, is such that in order to meet the emergency conditions, described below, any two sets working in parallel are capable of continuously supplying sufficient pressure oil for the operation of the governor gear of three water-wheels.

(*c*) Each oil-pumping set comprises a motor-driven rotary oil pump of the gear wheel type ; an oil pump fitted with water-cooling coils and with duplicate removable oil filters ; an air-vessel fitted with oil level gauge, oil pressure gauge, automatic oil level adjusting device, unloading valve and relief valve ; and all necessary piping and valves between the pumping sets in the auxiliary gallery and the water-wheel governor in the machine room. Separate oil filters are provided for filtering the supply of oil required for the operation of the unloading valve, of the needle-locking valve, of the deflector locking valve and of the servomotor for the automatic valves described below.

(*d*) The motor driving the rotary oil pump is of the squirrel-cage induction type wound for 400-volt, 3-phase, 50-cycle, and is provided with approved starting gear.

(*e*) Any two oil-pumping sets are capable of maintaining the supply of air necessary for 3 air-vessels. The level of oil in each air-vessel is automatically kept constant by having small orifices, one in the air-vessel discharging into the sump, and one in the pump suction pipe in the sump ; each orifice being arranged at the proper level of oil in the air-vessel and sump, respectively. With normal circulation of oil from the sump into the air-vessel, and from the air-vessel into the sump, the respective pump delivers into the air-vessel sufficient air with the oil to maintain the requisite quantity of air. When the oil in the air-vessel is below its proper level, the level of oil in the sump is correspondingly higher, thereby closing the air-orifice in the pump suction pipe. At such times the orifice in the air-vessel, by discharging only air into the sump, allows the pump to deliver the requisite amount of oil without air to raise the level of oil in the air-vessel.

Similarly, when the oil in the air-vessel is above its proper level, the level of oil in the sump is correspondingly lower, and thereby allows the pump to deliver sufficient air with the oil to adjust the level of oil in the air-vessel. The orifice in the air-vessel is controlled by a non-return valve, which, by closing at a pre-determined pressure, lower than normal, prevents the air-vessel losing pressure when the pump stops.

(*f*) All four pumping sets are inter-connected by means of a common piping system, so that in the event of the failure of pressure in any one of the oil-pumping sets whilst on load, provision may conveniently be made for the following :—

(1) The immediate and automatic isolation of that pumping set from its respective water-wheel governor and servomotor and also from the common inter-connecting system.

(2) The immediate and automatic supply of a limited amount of oil to the water-wheel governor and servomotors affected by this emergency from the air-vessels of any pumping sets, which may not be running at the time, but whose air-vessels have normally been maintained fully charged for this purpose.

(3) The complete and continuous supply of oil to the same water-wheel governor and servomotors, preferably from anyone

of the *stand-by pumping* sets, referred to in the previous paragraph, as soon as it can be started up after the sounding of the "*alarm*," described in clause 2, or, alternatively, from any two pumping sets already in service, which are made to work in parallel by the hand operation of suitable valves.

(*g*) The inter-connecting piping system includes an air header, a pressure oil header, a return oil header, a drain oil header, a drain tank and all necessary branch pipes and valves. Portable filtering plant is provided as part of the alternator lubricating system (*provided by the B. T. H. Co.*), but all necessary branches and valves for the ready connection of the filtering plant come under this main heading of water-wheels (*supplied by Boving & Co.*).

Isolating valves are provided at all branches on the pressure oil header and return oil header, and also elsewhere as necessary to permit of the independent drainage of any governor oil pumping set or oil header. An overflow pipe is provided from each pumping set to the drain oil header.

(*h*) Also in order to meet the emergency requirements, described in sub-paragraph (*f*) on the failure of oil pressure, *a control valve of the positive action trigger type* automatically operates and causes a servomotor to *shut* two automatic isolating valves, one on the pressure oil side, and one on the return oil side of the faulty pumping set, thereby isolating the set ; simultaneously the same servomotor automatically *opens* two other similar automatic isolating valves, one on the respective branch pipe from the pressure oil header, and the other on the respective branch pipe from the return oil header, this directly connecting the governor and servomotors affected, to the pressure oil and return oil headers, through which they receive, by the means described below, a proper circulation of oil from one or more *healthy pumping sets.*

A small bore hand-operated valve is arranged to permanently by-pass each of the automatic isolating valves and thereby maintain restricted connections from the pressure oil and return oil headers to the air-vessels and oil sumps, respectively, of all four pumping sets, whether the latter are in service or not.

These by-pass valves, by working in conjunction with suitable inter-connecting air pipes, thus maintain uniform oil pressure

and level throughout all the governors and pumping sets, and, in emergency, also permit of the limited supply of oil to the affected water-wheel governors and servomotors referred to in sub-paragraph (*f*-2) above.

A full size hand-operated valve is arranged to by-pass, when required, each of the automatic isolating valves of any one or more pumping sets, and thereby establishes full bore connections from the pressure oil and return oil headers to the respective air-vessels and oil sumps.

These full size by-pass valves are normally kept closed, excepting those pertaining to stand-by pumping sets, which are kept open so as to give the complete emergency supply of oil described in sub-paragraph (*f*-3), immediately the stand-by pump is started up.

A hand-operated isolating valve is provided in each of the pipe connections between the automatic isolating valves and the water-wheel governors and servomotors.

(*i*) Suitable blanked-off branches are provided on all headers for the addition of a spare pumping set, if, and when, required at a later date.

(*j*) All inter-connecting air-pipes have non-return valves and isolating valves where they take off air from the various air-vessels.

If required in the future, any one or more air-vessels may be quickly charged with air from an existing central air compressure by making a suitable connection at the extreme end of the air-header.

(*k*) Suitable provision is made at one end of the return oil header for the purpose of filling the governor oil system, but elsewhere all headers are terminated with blank flanges.

(*l*) In order to facilitate identification and operation of all valves at site, each separate valve on any one set is provided with a metal label or disc bearing a distinctive, permanent and easily distinguishable number and the corresponding valves on all four sets are similarly numbered. For the same reason pipes are painted, after erection, with two coats of the following colours :—

Pressure oil pipes	Post Office red.
Return ,, ,, ...	Dark blue.
Drain ,, ,, ...	Black.
Water-cooling pipes	Light blue.

6. Breaking Jets:—Each water-wheel is fitted with a *breaking jet* to bring the machine to rest, as soon as possible, after the main jet has been deflected or interrupted by the closing of the needle. The nozzle of the breaking jet is of stainless steel. The supply of each breaking jet is brought by means of the pipes from the upstream side of the main inlet valves, but only one branch pipe is used to supply two jets, one on each of the two adjacent machines. The jet is on that side of the water-wheel which is remote from the inlet bend, *i.e.*, just opposite the main jet, but the *needle valve* for controlling the breaking jet is arranged on the inlet-side of the water-wheel on one side of the bed-plate.

7. Tachometers:—Each water-wheel is provided with a thoroughly reliable tachometer by an approved maker, forming an integral part of the governor with a dial of 12-inch diameter and of *Horn's type with ball bearing*, marked to every five revolutions and reading up to 860 revolutions per minute. The tachometer is fixed in such a position as to be readily seen from the governor hand-wheel and the *normal speed marked with a red line*.

8. Water-Pressure Gauges:—Each water-wheel is provided with a water-pressure gauge of *Bomdon type*, so connected as to measure the effective head acting on the water-wheel.

The gauge is mounted directly on the top of the inlet bend and is sloped backwards so as to be easily visible to an operator standing.

9. Care and Maintenance:—The scale of the indicator (108) should be marked to show the guide vane opening in mms. or cms.

The governor should be thoroughly cleaned; all bolts, key-ways and screws must be carefully tightened and locked. It should then be filled through (120) with *clean filtered oil* to the top of inspection window (118). When in service, the oil level should be about half-way up *gauge glass* (123) and the oil in reservoir (1) will then be at the lower edge of window (118) as in the installation. The oil *must always* be filtered before filling.

The hand-control on the governor should always be coupled in when the turbine is not running. The belt for the governor-drive should be thin and flexible, preferably of the endless type. A good quality three-ply "*Balata*" belt is usually satisfactory, but the "*Balata*" surface should not be in contact with the pulley. Care should be taken that the belt is straight and on no account should belt-wax be used. It should be kept fairly tight and it may be necessary, after the belt has been in use for sometime, to slightly shorten it owing to stretching.

To make a satisfactory joint, if an endless belt is not used, belt fasteners of the " Bristol " type No. 1 may be employed and these should be bent with a hammer to fit the diameter of the smaller pulley so as to reduce the shock caused by the passage of the joint over the pulley.

Governor Oil :—The oil used in the governor conforms to the following specifications :—

Viscocity at 20° C., 20° Eugler or 616° Redwood, at 50° C 121° Redwood.

The tar content less than 1%.

The flash point = 170° C.

Mineral oil is used and not blended oils.

Coupling Governor In or Out

When putting in the mechanical hand-regulating gear, the hand-wheel (80) should be turned slowly to allow the teeth of the worm (77) to fall into mesh with the worm wheel (74). At the same time the hand-lever (79) should be pulled out and then quickly turned *downwards* as far as it will go, the turbine will then be on hand-control.

In order to disconnect the hand-control the hand-spoke must be pulled out and turned in the upward direction about half-way until the pressure in the pressure chamber commences to fall as evidenced by the pressure gauge thereon. It should be held in this position for *several seconds* until the servomotor cylinder has become filled with oil and the pressure in the pressure chamber has again become normal ; when it should be turned quickly upwards to its full extent, the turbine will then be on *automatic control.*

Starting Up of Governors

See first that there is plenty of oil in compensator, also that the hand-control is coupled in and that the guide apparatus of the turbine is fully closed. The water then can be admitted to the turbine, and in case where it is supplied through a pipe line, and *especially when this is long,* care should be taken that the water be admitted gradually and without shock. The intake gate at the head of the pipe line should be opened a small amount at first so that any air present can escape. If the water is admitted suddenly, water hammer will result which may possibly cause the destruction of the pipe line. When the water has been admitted to the turbine and after careful inspection has shown that

everything is in order, that all the bearings are well-lubricated and grease-cups, etc., filled, the guide apparatus may be opened slightly by means of hand-gear until the turbine begins to rotate. The speed should then be *gradually* increased to normal.

Should there be no pressure in the pressure chamber (2), the valve (102-106) should be partially opened in order to prevent the pump from pumping the whole of the oil into the pressure chamber. When the speed of the turbine is approximately normal, the *air-valve* (13) on the suction side of the pump should be opened and sufficient air pumped into the pressure chamber with the oil so as to force the oil level to the centre of the gauge glass (123) when the normal pressure is registered on the gauge (103). The valve (102-106), which, at the commencement of the operation, should have been open, must now be tightly shut, having been *gradually closed* as the pressure in the air vessel approaches normal. The air valve (13) must now be closed. When the oil-pressure and the speed of the turbine have become normal, the wheel (81) should be slowly twined so that the indicator (45) comes directly over (46), as shown in the illustration.

The turbine can be put on to automatic control, as previously explained.

The oil-pressure is stamped on the name plate, but this need not be higher than is necessary to effect satisfactory opening and closing of the turbine gates.

Governors for Turbines with Long Pipe Lines

When starting up turbine with long pipe lines, great care must be taken not to set up water hammer which might burst the pipe line. For such cases the governors have the openings of the *regulating valve* limited in size.

Load the turbine by means of a water resistance up to ¼ load, then throw off load suddenly and observe the pressure rise. If this is in the neighbourhood of ¼ of the maximum permissible pressure rise, it will be satisfactory to proceed without alteration, making the same test with ½ and ¾ and finally full load. Should the pressure rise observed be considerably below the maximum allowed, the hole or groove in the *closing* side of the valve (40) can be enlarged slightly. With long closing times the enlargement should be very small, at the most 1/10 mm. at a time. A pressure rise of 50% of the head is generally allowed, when full load of all the turbines connected to the pipe line is suddenly thrown off. (See the turbine description).

Note :—Before altering the "*timing*" above, all the turbine control mechanism should be examined to ascertain that there is no *abnormal friction* in any part.

The oil-pressure and *temperature* must also be at their normal values, before the tests are carried out.

Alteration of Speed :—The screws in the pendulum cover are taken away and the cover of the pendulum is removed. One can then see the spring holder with the pendulum spring. In the spring holder, there are three adjusting screws. If the adjusting screws are turned to the right (clockwise), the spring holder is raised, the pendulum spring thus extended and the pendulum (the turbine) runs with a higher speed. If the adjusting screws are turned to the left, the spring tension is decreased and the pendulum (the turbine) runs at a lower speed.

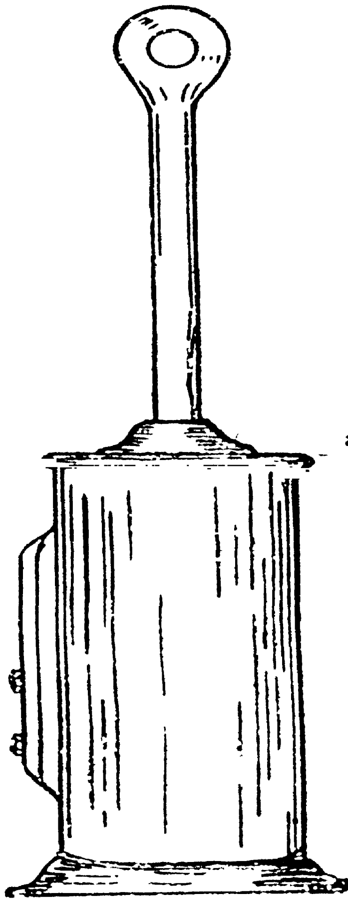
The pendulum spring and the spring holder have a red mark and the spring casing is also marked in order to indicate the original position. The speed set at the works ought not to be altered, if possible. Care is taken that the upper edge of the spring holder is parallel with the upper edge of the spring casing.

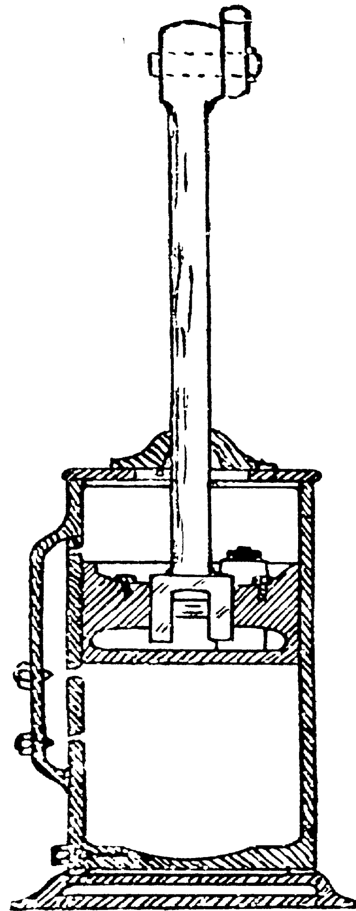
At the correct speed of the turbine, the pendulum rod will be near its middle position, *i.e.*, at the middle of the whole stroke of the pendulum rod.

Alteration of Speed Range :—The speed range is increased by shortening free part of the pendulum spring. The adjusting screws in the spring holder are then screwed to the left so that the pendulum spring is slackened off. A pair of the adjusting screws are completely unscrewed right out, then reversed and screwed in the holes. The spring holder is turned holding the adjusting screws well-gripped so that the spring is screwed on the spring holder. The spring holder with the spring is then turned back a little so that the end of the spring is at right angles to the cross-head. The adjusting screws are reversed again and screwed in until the spring becomes stretched.

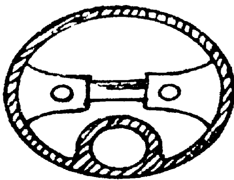
The speed range is decreased by lengthening the free part of the pendulum spring, *i.e.*, by unscrewing the spring from the spring holder. This is done in the same way, as described above, with this difference that the spring holder is turned to the left. Care is to be taken that the end of the spring is at right angles to the cross-head and that the spring does not touch any of the adjusting screws. The cover is put on and the pendulum started. If the speed is not right, it is adjusted, as already indicated above.

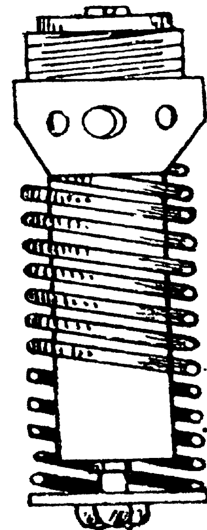
Fig. 17



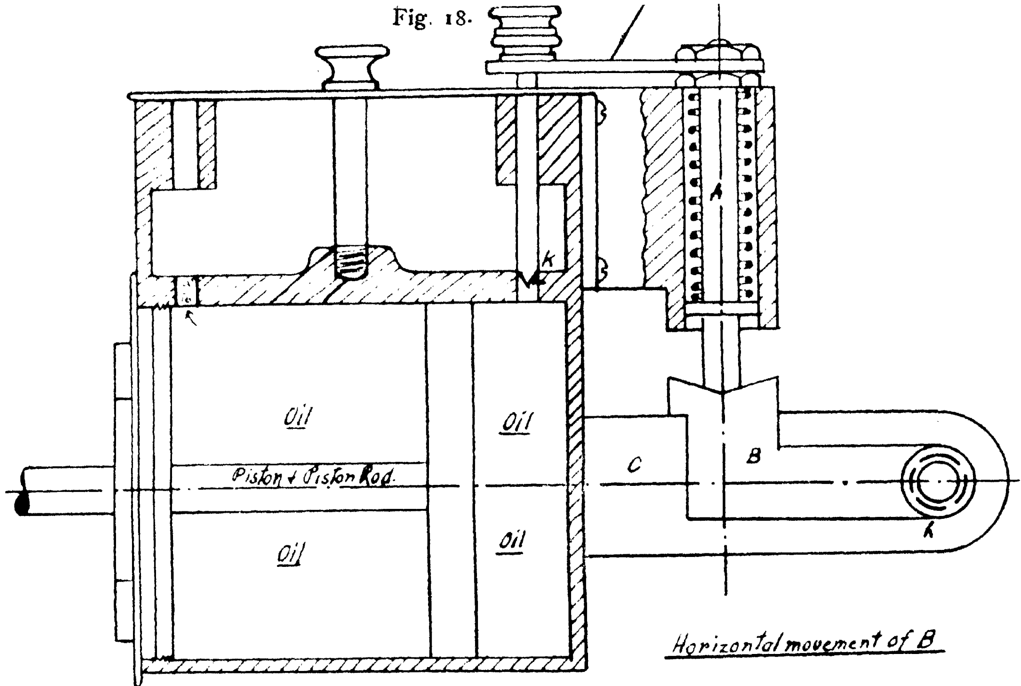


Dash-pot arrangement at Pykara.





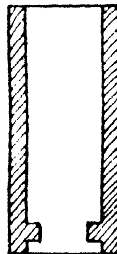
For Dash-pot arrangement at Mahora (see back)



Horizontal movement of B lifts the rod A & thus regulates the opening at K through the needle

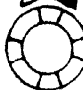
The movement of B is due to the hole "h" being bigger than BB in B

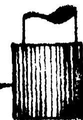
Dashpot – (in section)



Cylindrical Ring in which the threaded Rod Works to regulate the height of the governor according to load







This gears with a rack

Rack and pinion arrangement.

Compensator

The compensator (Fig. 15) consists of a cylinder and piston in one piece in a valve which controls the passage of oil between both sides of the piston. The compensator is hydraulically coupled to the servomotor piston in such a way that when the servomotor piston moves upwards, the compensator piston is pushed in the direction towards the pendulum, and when the servomotor piston moves downwards, the compensator piston is dragged in the direction away from the pendulum, the motion being in both the cases transmitted by pressure and suction of the oil. These movements of the compensator piston are always in the same direction as those of the regulating valve piston and the pendulum regulating pin. When the turbine speed drops, these pins move towards the pendulum and open communication between the upper side of servomotor piston and discharge ; when the servomotor piston moves upwards and presses the compensator piston, the communication with the discharge is closed. The spring packing of the compensator then brings the compensator piston slowly back to its mid-position.

Dash-Pot

With load changes of greater magnitude tending to cause a high rate of movement of the 'regulating shaft' referred to (which controls the gate mechanism by links and which is controlled by the pendulum, etc.), the dash-pot (linked to the gate-operating rod) extends by a downward force on the rod in proportion to the rate of gate-opening (Fig. 16). The movement of the rod above a predetermined limit will be checked by the opposing force due to compression of the dash-pot spring. This will restore the rod or gate-opening link to its normal position arresting the movements of the gates in advance of the restoration to normal conditions. The fulcrum then returns to its normal position in a certain time which may correspond to the time it takes for the turbine to respond to the change in gate-opening due to the new load conditions.

The governors are equipped with mechanical hand-control independent of the servomotor. There is a mechanical hand-regulating gear consisting of a worm-wheel and worm, the spindle of which is guided in an eccentric bearing so that it can be brought in or out of mesh by turning a handle. The bearing is at the same time a plug of a cock through which the pressure side of pump is connected to the oil tank when the worm is in mesh so

that the pump runs light. There is also a hand-control of the pressure only usually called 'governor control.' This control is independent of the centrifugal speed element and is of great value for adjusting the load on the unit and for synchronising purposes. This control can also be operated by operating a small reversible motor (electric) the shaft of which is geared to the control wheel.

Troubles and Remedies:—All governors are adjusted at the works of Verkstaden, Kristinehamn (Sweden), but it may happen that during transport and erection, the setting is altered in such a way that the automatic regulation is not satisfactory.

Hunting:—The governor shaft moves to and fro whilst the load is constant. Before investigating the governor it should be ascertained whether there are any oscillations in the open flume or the pipe line which may be caused by other turbines, and which would result in oscillations of the governor For this purpose the governor should be put on hand-regulating gear, and the turbine speed observed by means of a *hand tachometer* to see whether there are vibrations in speed in spite of the load being steady. It is possible also to use for this the tachometer mounted on the governor, but as this is affected by the irregularities of the belt drive, it is only dependable if the drive is quite correct. The following are possible causes of hunting :—

(*a*) The *valve* (41) moves sluggishly. It should be checked that the cover (43) has been placed on evenly. The screws on this cover should be tightened in pair diametrically opposite. The friction of this valve must be an absolute minimum; it should be capable of descending into the piston (42) by its own weight, when the latter is held vertically.

(*b*) *Air in the Compensator*:—This is frequent cause of hunting and the air must be removed, as described before. This operation must *never* be carried out when the turbine is on automatic control.

(*c*) Faults peculiar to the compensator are generally caused by *unsuitable oil*, dirt or by the valve (42) sticking. Ascertain that the small holes in (67) under (68) and (69) are clear. The compensator has been previously described under heading "*Compensator*."

(*d*) It should be tested whether the belt-fasteners and belt-pulleys are quite round and also that the belt itself is quite straight and not too tight or too loose. The faults with regard to the belt do not cause hunting

proper, but result in a "*jumping*" movement causing undue wear in the governor and control apparatus.

(*e*) Check whether the pendulum speed drop (*i.e.*, the difference in R. P. M. for the two end positions of the *pendulum pin*, multiplied by 100 and divided by the normal pendulum R. P. M.) corresponds to that stamped on the pendulum casing. By increasing the speed drop, which may be obtained by using fewer turns of the spring (38), the pendulum will become more steady, but at the same time the momentary speed variations will be increased.

Should the Governor close the Turbine in spite of the Speed being too Low:—This may happen when the governor is being put into operation and the oil is cold and very thick. After the governor has run for an hour or so and the oil has become warm, it should work satisfactorily. If, however, this fault should occur after the oil is warm, it is caused by dirt entering the regulating valve and chocking one of the canals. The valve should be dismantled and cleaned. *Do not use a furry cloth* for this purpose and wash the valve carefully afterwards with paraffin. In a governor with *a long closing time* the regulating valve is extremely sensitive to dirt owing to the small openings necessary.

Should the Governor Open the Turbine in spite of Speed being too High:—Caused by dirt in the regulating valve or by the governor belt slipping.

If the Speed does not go back to normal after a change in Load:—Caused by friction, wear or by dirt in the compensator, causing the latter to stick. It may also be caused by the turbine control mechanism becoming rusty or by foreign material being lodged in it. It can also be caused by nut (58) being loose.

Should the Pressure in the Pressure Chamber Fall Excessively:—Caused by dirt in *valve* (106), non-return valve (98) or valve (93) by unsuitable packing or by an *air-leak*. In the latter case, the level of the oil on the indicator glass (123) does not necessarily fall although the pressure does. The pressure will fall slightly after stopping the turbine. This is due to the air in (2) becoming cool.

Note:—When the servomotor piston (4) moves, the oil level in the air vessel will temporarily sink a small amount.

If the Unit Runs in Parallel with Other Machines and the Generator does not take its Share of the Load:—This can be adjusted by means of the eye-bolt (62) which connects the

compensator to the return gear. If the turbine takes too much of the load, the movement of (60) should be increased by moving (62) outwards in its slot.

Note :—An incoming machine has a tendency to take too large a share, as its oil becomes warm. In this instance a correction can be made by means of hand-wheel (81) or motor (85). For isolated units eye-bolt (62) is adjusted so that no movement is passed on to (60).

This only applies to units which do not run in parallel with other sets.

If the Unloading Valve Chatters :—This is perhaps due to the strainer (99) being dirty.

If the Oil Should Froth Excessively :—This can be caused by an air leak on the suction side of the pump pipe (114), continuous leakage from valves (98 and 106) or by unsuitable oil.

Note :—A certain amount of frothing occurs when the pressure chamber (2) is being charged, but this will cease after the operation has been completed. If valve (13) is left open, the oil will froth. Continuous frothing *must not* be allowed.

Lubricate the *tachometer* with best quality sewing machine oil.

The quantity of air in the pressure chamber is adjusted by slightly opening (122) if too great, or by opening (13) if too small. Both must be closed tightly when the quantity is correct, (see "*Starting up of Governors*").

The filter (99) for the unloading valve must be cleaned as may be necessary.

After the governor has been in operation about one month, all oil should be emptied and the governor thoroughly cleaned and washed with paraffin, none of which should, however, be allowed to remain in the oil tank. The governor should then be filled with new oil or with the old oil after careful filtering. *Do not forget the filter when filling.* This complete dismantling and cleaning should afterwards take place once a year, all parts being inspected and cleaned, the nuts, keys, etc., checked and the oil filtered and replenished. The *regulating valve* itself should be dismantled and cleaned once a month.

When dismantling the governor, attention should be paid to how the parts fit together, and if any adjustment is altered, new marks should be made or a record kept so that the parts may be re-erected correctly.

Internal parts should be washed with paraffin, and if dried, this shall be done with *a cloth which does not fluff. Waste must not be used.*

The belts are tightened when they begin to stretch. Any coating on the pulleys should be cleaned off.

Some of the Recent " Hydraulic Troubles " in the Glen Morgan Scheme at Pykara

Governor Troubles :—(1) Oil was found dirty. The oil was changed. The operating shaft was found badly rough, and it was filed and polished. The other parts were also cleaned.

(2) The governor was one day hunting at the time of " off load." The control valve was opened and cleaned and fresh oil put. The pendulum rod was slightly bent. The rod was turned and refiled. The governor was still found hunting. So it was again opened. This time the fish tail plate of the governor pendulum spring was found bent, and it was adjusted properly. Everything was O. K. afterwards.

(3) The same trouble on another occasion—hunting during " off load." Opened the governor. One leather washer on the fixed plunger was found damaged. The rail hole was not opposite to the corresponding holes through the thread. Fresh leather washer was put in and the hole set in properly. The control valve was also cleaned. Started the machine O. K.

(4) Yet another instance of the governor hunting at the time of cutting off of load from the machine.

On opening, the control valve was found sticking to the sides in some places. It was set aright by delicately rubbing with the file and polishing with emery. The valve was cleaned and refiled. Tested at load, the governor was still hunting. The governor was again opened and on examining it was found that the check nut holding the leather washer on the fixed plunger not screwed down properly. The same was set aright and the parts re-assembled. The dash-pot was cleaned and filled in with fresh oil. Tested O. K.

(5) Another kind of trouble was the falling off of speed in spite of adjusting the regulator. The governor was completely overhauled. The servomotor piston was found stiff and, therefore, cleaned with emery. The relief valve oil-pressure was adjusted by two turns and raised the pressure to 210 lbs. per sq. inch. The oil and packings were renewed. Tested O. K.

(6) Another nature of trouble was the wrong indications of the tachometer mounted on the machine. The actual speed was

found by means of a hand tachometer of the sensitive type. The readings compared thus :—

Revolutions Indicated.		*Actual Revs. (Standard Inst.).*
750	... ...	765
725	... ...	742

After testing the tachometer properly the machine was brought up to normal speed and put on governor control. The speed was 765. Then keeping the throttle in neutral position, the pendulum spring was slackened till the normal speed was indicated on the machine.

(7) The governor belt was slack ; this was shortened by 1″ and restitched. The tachometer belt also was shortened and restitched.

Various electric troubles are in fact traced to the improper functioning of the governor. One was :—

(8) Station voltage falling down, the speed of the unit going down to 700 from 750 R. P. M. The oil-pressure of the governor was normal. Regulated the governor control, but to no use. Put the machine on hand control and adjusted the speed. It was O. K. Tested the governor disconnecting load and found the servomotor piston sticking.

(9) Governor refused to act in spite of the speed being very low. The oil-pressure in the governor was *low*. The relief valve was slack. Adjusted it. Everything was O. K.

(10) Observed heavy variation of voltage and the machine No. 3 threw off the load suddenly and the speed began to drop. Regulated the speed of the machine on the throttle, but to no good. Hence, took the machine on hand control and brought up to normal speed and resumed load in about a minute. The trouble must have been due to spear sticking.

(11) Bad hunting of both the machines when synchronised. The unit at load was O. K. The trouble, perhaps, was due to the oil in the governor of the incoming unit being too cold or dirty.

Nozzle Gear and Dash-Pot Troubles :—Nozzle stuck : Needle would not close though governor acted. Shut down the machine by closing the main valve and lubricated the needle journal and cleaned the dash-pot.

Deflector 12 mm. from centre line when closed.

Deflector 41 mm. from centre line when open.

Once, just after the time of peak load when the load was going down, No. 3 unit, which was working in parallel with No. 2,

behaved very badly. The turbine was making a peculiar heavy noise with a heavy discharge of water to a length of 20 ft. in the tail race. The heavy discharge stopped, however, and the machine kept on. The same behaviour was observed the next day also almost at the same time. The cause ?

It may be due to :—

(1) the sticking of the dash-pot, or

(2) the sticking of the nozzle caused by improper seating of the cup leather packing of the nozzle.

The dash-pot was opened and on examination found the plunger sticking to the sides. This was rectified and the dash-pot refilled and fresh oil was filled in. The machine was on load throughout the night and behaved O. K.

Penstock Valve and General Troubles :—The main valve would not shut off water completely. It was suspected that some foreign matter had entered the bottom of the valve seating. Tried opening of the nozzle as far as possible without any effect. Emptied the pipe and took out the valve. The seating on the face side was broken and badly jammed. The broken portion of the seating had jammed the needle also. The needle was put right and the valve closed with the spindle above and the machine was kept for emergency use.

After 2 or 3 days shut down the plant and replaced the valve which was repaired in the workshop. A gun-metal seating was cast and screwed down to the valve base with copper screws and riveted and turned to the requisite thickness. The valve made a water-tight joint.

Another trouble was a bad leak of water at the top joint of the drain valve due to the packing at the joint giving way.

There was a general shut-down for 3½ hours. Emptied the pipe line, renewed the drain valve and put in fresh packings (Klengerite packings with red lead) at the joint and refiled the valve. Also fresh tarred paper packings were put at the top and bottom joints of bent portions of the drain pipe. Filled in water in the penstock and found the joint O.K.

The regulating apparatus in connection with conduits and tunnels for storage dam outlets, which may be used, are generally of four different arrangements :—

(1) A conduit through the dam with a regulating valve installed at the entrance to the conduit on the upstream face of the dam. Such a valve usually consists of a needle-shaped plug pointing towards the mouth of the conduit and enclosed in a

cylindrical structure form or set into the face of the dam. The conduit leading from the valve receives the water at spouting velocity and carries it through the dam to the downstream face. The valves are submerged and inaccessible for maintenance and repairs unless it is possible to draw down the reservoir and unwater them.

(2) A supply well is provided in the dam structure in which water is admitted through slide gates and from which it is discharged through a conduit having a regulating valve located at the entrance to the conduit. The slide gates are located at several different elevations in order that they may successfully operate under low pressure as the water-level in the reservoir is drawn down.

(3) The same as in (2) except that the regulating valve, located at the discharge end of the conduit, discharges directly into the air.

(4) A conduit through the dam with a regulating valve located at the discharge end of the conduit on the downstream face of the dam.

Whereas the valve located at the downstream end of the conduit is accessible for inspection and repairs, the valve at the upstream end is never accessible except when it is possible to draw down the reservoir. If a valve gets out of order, it cannot be repaired until the low-water season. For this reason it is customary to place batteries of valves at different levels to be sure that some of them will be regularly unwatered by the seasonable variations of level in the reservoir. Furthermore, the upstream type of valve cannot be protected from debris and, on account of the high velocity at which water is drawn into it, it is very likely to get plugged in such a way that the valve will be put out of commission.

CHAPTER XIII

POWER-HOUSE AND STATION LAY-OUT

Purpose of the Power-House :—This is to support and house the hydraulic and electric equipment, to shelter and protect the machinery and operators and to provide facilities for handling the equipment, in case of hydro-electric stations.

Location of Power-House :—Its location is determined by the development programme. It may be at the end of the dam or inside of the spill-way immediately below or at the terminal of diversion canal or pipe line (*vide* Fig. 1). In the first position it may receive the water directly from the upper pool or from a forebay guarded by head-gates. When taking water from the pond, the power station stands on the dam alignment; when fed from a forebay, it stands at some angle to the dam, the forebay paralleling it on the land side. In either position the power station is exposed to all the consequence of the fluctuating head and flow and, when on the dam alignment, to those growing out floatage accumulations.

Type :—This depends upon the head. For low-head developments it will generally be located at the spill-way or closely below it, with a medium head, from 30 to 60 feet; the power-house may be at the spill-way or at the end of the diversion works; while for high-head developments it will always be at the terminal of a pressure line.

Nature of Passage of Water into the Power-House :—The height of the fall, the topography at chosen power-house and the volume of the flow determine whether it is recommendable to let the water enter the power-house freely or by means of feed pipes.

*In designing power-house the following factors should be carefully considered :—

(1) *Natural conditions* :—(*a*) proximity to streams, (*b*) the condition of soil, etc. In hydro-electric stations—(2) the type of turbines, which depends upon the nature of the head—the construction of the turbines—whether horizontal or vertical. (3) The number and capacity of the units. Further, if the location of the power-house is far away from the point of distribution, thus requiring high-tension transmission, there must be provision for

* *Vide* Economics of Engineering, Ch. XII, by Coueslant and Chatterjee.

Fig. 1.

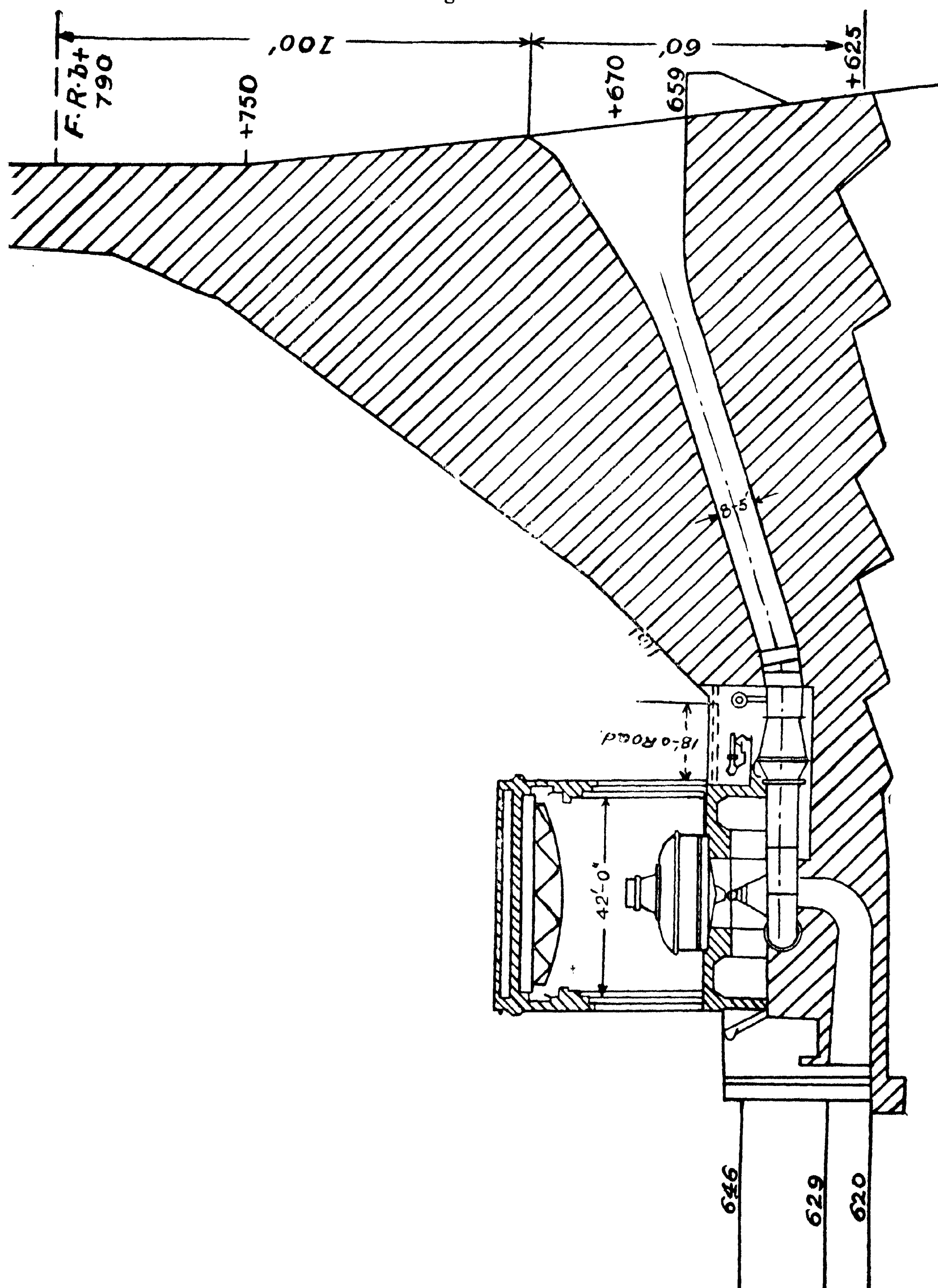
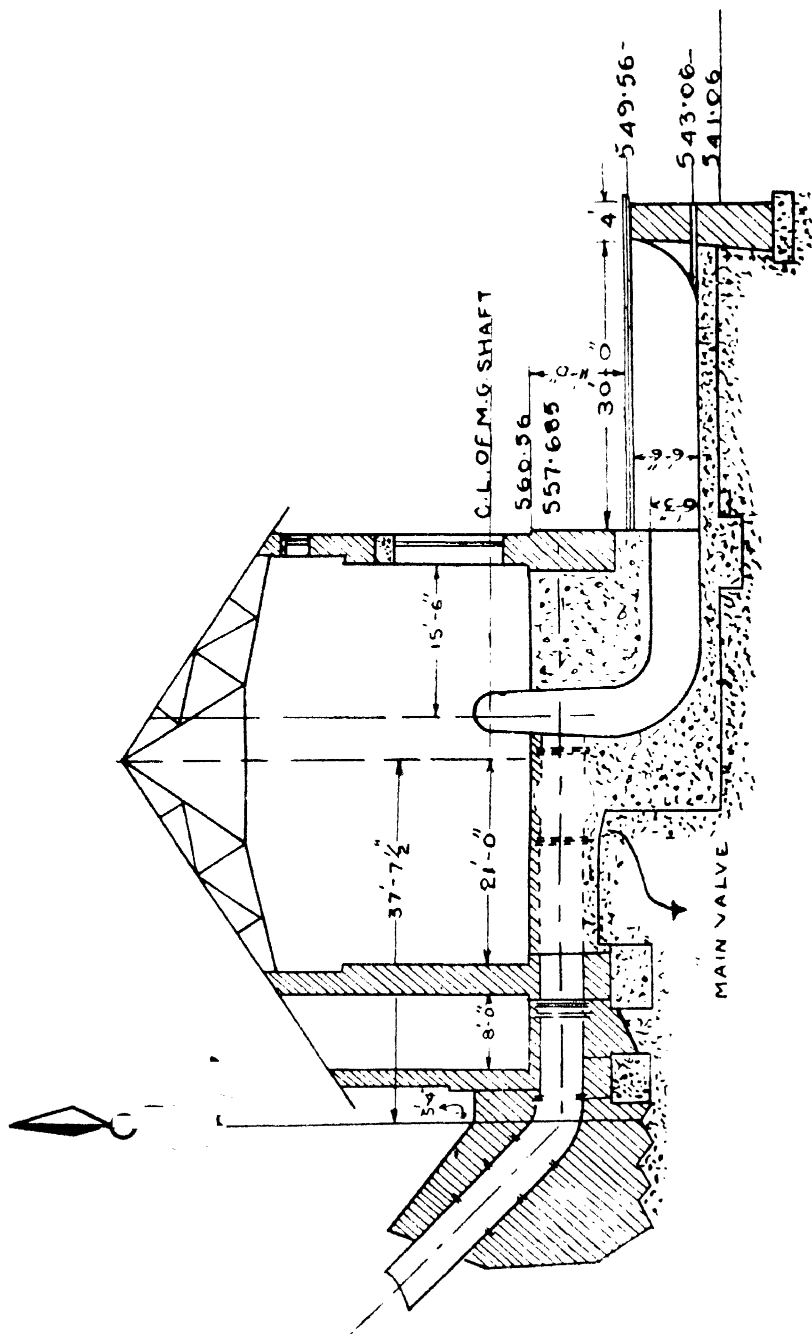


Fig. 2


CROSS SECTION OF GENERATING STN.
SCALE 1" = 20
C.L. OF M.G. SHAFT
560.56
557.685
549.56
543.06
541.06
MAIN VALVE
15'-6"
37'-7½"
21'-0"
8'-0"
30'-0"
4'

housing the transforming and high-tension switching apparatus and in this case whether the transformers are installed outdoors or within the power-house.

The arrangement of the apparatus should be given the first consideration, simplicity in design and harmony with the surroundings should also be taken into consideration.

The Elements of the Structure :—In hydro-electric plants, however, there are three distinct sections :—(1) The foundation of the substructure which supports the apparatus such as the turbines, etc., and provides the necessary waterways. The power station rests upon a substructure whose upstream part, excepting when water enters in pipes, fills the office of a dam. (2) The pit and the tail race forming the exit passage. (3) The superstructure houses protect the apparatus from the elements and support the travelling crane which is used to handle the equipment.

(1) **Foundation** :—This must adapt itself to the character of the material at the site. The foundation considerations are the same as those described for the spill-way, as the power-house, in a sense, fulfils the functions of a dam. The cutting-off of substrata by which water might pass under the foundation, is of vital importance; in fact, the power-house foundation in an alluvial location is best entirely surrounded by a substantial cut-off wall, the foundation proper overhanging the cut-off at least three feet. The sliding of the structure, under the pressure from the upper level water, must be likewise considered and guarded against, and an apron should be placed along the entire downstream side, being from 8 to 16 feet wide according to the depth of the water in the tail-race.

(2) **The pit and tail-race** are part of the power station, forming the exit passage of water discharged from turbines. The first is arranged in the substructure of the station, the second in the river channel below the power station. No serious defects are to be expected in either of these when, by proper design, they are of ample area and depth. When the equipment consists of several units, some of which are operated only for the generating of secondary power from higher than normal flow, it may be found that the tail-race under and from the idle units becomes silted, thereby reducing the depth under draft tubes to such an extent that the outflowing water backs up and reduces the effective head. The ready remedy is to operate each unit alternately to avoid long periods of idleness.

(3) **The superstructure** must accommodate the power equipment.

Wherever permissible, the water should be taken to turbines in conduits, as the power-house required for such an arrangement will be considerably less costly than where the water enters freely.

It is otherwise recommendable that the power-house units should be rather large than small, as the length of the power-house materially depends upon this condition. Where water enters freely, the structure performs, in a sense, the functions of a dam, and the foundation and pit structures required in that case must be designed to (1) support the vertical loads, and (2) to resist horizontal pressure. These add considerably to the dimensions.

Dimension of the Building :—The lay-out of the power plant equipment including turbines, generators, exciters, switchboard, oil switches, and other auxiliary machinery, sometimes high-tension transformers, determine the general over-all dimensions of the building. As a rule, the clearance of the travelling crane, necessary to handle the largest pieces of equipment and carry them over the other machinery, while it is in operation, determines the *height of the building, e.g.*, where the head gates, located inside the power-house and lifted by the crane, are the controlling influences.

Station Lay-out :—Although the total space occupied does not vary much with the character of development, it has a direct bearing on the general outlay adopted. High and low head developments require different types of turbines, and horizontal or vertical construction requires entirely different lay-outs. Thus, when in a hydro-electric station a *low head is utilised*, the space available is usually strictly limited by the width of the dam on which it is built. This, in its turn, is determined by the space required by the turbines. If the *head is high*, the station site can be usually so selected as to have ample ground space. The space being small, the switchgear and transformers are generally located either on floors above the machine rooms or in entirely separate buildings on the river bank.

Location of Apparatus :—The arrangement of apparatus should be simple and ensuring reliability of operation. The purpose of the station is to give reliable service. The causes of disturbance and means of minimising their effect must be carefully considered. The abnormal or so-called emergency conditions and the failure of every piece of apparatus should be anticipated and a definite plan should be worked out for limiting the .magnitude and area of such disturbances.

Location of Generators :—The generating sets are placed in row down the length of the station so as to facilitate handling by the travelling crane. The turbo-generator units are located on the main floor and are almost always arranged in a long line along the long axis of the station. They should be spaced far enough apart so that ample space for passage is provided between them. Horizontal sets may be installed either at right angles or parallel to the long axis, the latter method being necessary for high heads where impulse wheels are used. The arrangement of the rest of the equipment, such as the transformers, may also be a determining factor, in regard to which direction a set should be installed. If one transformer bank, consisting of single-phase units, is to be installed for each generator, the space occupied by them may be of such a length that it should be more economical to install the turbo-generator sets parallel to the long axis, thus reducing the width of the building.

Location of Turbines :—In horizontal shaft sets high-head plants using impulse-type water-wheels have their shafts usually parallel and other shafts are at right angles or parallel to the axis of the station. With horizontal sets the turbines may be located together with the generators in the generator room or in separate wheel chambers built in the dam. The latter practice is only used for very low-head developments, where one of the power-house walls forms part of the dam structure. With vertical units, the turbines are always located in a basement, the thrust bearing being supported on an intermediate floor below the main floor, unless suspension bearings are used, these being mounted on top of the upper generator-bearing bracket.

Location of Exciters :--If the generators have a common exciter, it should be central in relation to the generating sets and should be located as close as possible to the switchboard so as to minimise the copper in the exciter leads. This also applies to motor generator sets, when used for exciter. The system may be readily sectionalised, one exciter serving the generators located to one-half of the station and the others, the generator on the opposite side. The exciters are, as a rule, installed in the same floor as the main generators and in the center of the station. The advantages of such an arrangement is that the exciters will be located close to the operating switchboard and the amount of copper required for the exciter leads is thus a minimum. The system may readily be sectionalised, one exciter serving the generators located in one-half of the station, and the other, the

generators on the opposite side. This does not, of course, refer to direct-connected exciters.

Location of Switchgear and Transformers :—The transformers are generally placed outside the main buildings. Due to their weight the step-up transformers should preferably be located on the main floor. They are generally installed in isolated compartments in the rear bay, separated from the generating room by fireproof steel curtains. These compartments should be sufficiently large to allow a good ventilation. A car track is provided on the generator room floor in front of the transformer compartments where floors are raised so that the transformers can be run out on the car and moved to some convenient place in the station where repairs can readily be made. For large units it may be necessary to provide a hole in the floor above the repair room so as to enable the transformer core to be lifted out of the tank, or a pit may be provided into which the transformer may be lowered, so that sufficient head room is obtained for lifting out the core. Sometimes the repair room is so situated that the main crane cannot be utilised for dismantling the units. In such a case a chain fully supported from a heavy I-beam in the floor above may be provided.

Transformers Installed Out of Doors:—Considerable activity has recently taken place in installing transformers and associated high-tension apparatus outdoors. With the exceptions of the bushings the transformers for such installations differ comparatively little from the indoor type, the only feature out of the ordinary being the necessity of keeping the moisture from entering the transformer cases under the covers and leads. To prevent this the joints have been made with waterproof gaskets and breathing chambers have been provided.

Special precautions must naturally be taken to protect transformers of the outdoor type, both from the extreme heat and from the cold in the winter. The former can readily be obtained by providing sunshades, and in certain instances very good result has been obtained by simply painting the cases white. It is more difficult, however, to provide for the cold winter temperatures, especially with water-cooled transformers. With the transformers in service there seems to be no danger of freezing, and if such should be the case, some sort of heating grids could readily be provided in the bottom of the tanks. The main difficulty lies in the formation of the moisture which takes place

when the temperature of the transformer is allowed to fall below that of the surrounding air; this applies also to indoor transformers. Precautions must, therefore, be taken that this does not happen, and may be accomplished by either reducing the water rate at times of cold weather or by using the cooling water over and over again. Non-freezing oil may be used in such transformers, but its cost is so high that it is almost prohibitive from commercial stand-point.

Location of Switchboards and Switchgear:—The different pieces of apparatus comprising the switching equipment are distributed on the various floors in the switch section of the station, each story being partitioned to suit the various purposes. The operating room with the control switchboard is generally located on the second floor, and in such a position that the operator may have an unobstructed view of the station, and be able to readily communicate with the turbine operators. A balcony, somewhat overhanging a generator room in front of the switchboard, is often provided; or the operating room is built with a curved front wall extending out over the generator room.

Location of Oil Switches :—The low-tension oil switches are generally of the enclosed type and, together with the low-tension busbars, are located generally in the compartments on the main floor back of the transformer compartments. The switches themselves should preferably be set opposite the generator and transformer bank which they control, so as to call for as short a connection as possible and in order that these connections may be of equal length. The high-tension oil switches and busbars, and also, as a rule, the lightning arrester tanks, are installed on the floor above.

Disconnecting Switches:—It is customary to install disconnecting switches on both sides of an oil switch, so that they may be entirely disconnected from the circuit when repairs are to be made on them, when the oil tanks are to be refilled, etc. Disconnecting switches may also be used in a number of cases for changing connections when this is not to be made underload. Such switches should be provided with locking devices, as experience has shown that the magnetic field caused by short-circuit's may cause disconnecting switches to open, which, in turn, may cause serious disturbances by the arcs set up.

Spacing of Busbars :—The following spacing may be used in spacing busbar structures.

Voltage of Circuit	*Distance in Inches.*		
	Between centres of conductors of opposite polarity.	Min. between live parts of opposite polarity.	Min. between live parts and ground.
3,300	6	$2\frac{1}{2}$	2
7,500	9	4	3
15,000	9	5	$3\frac{1}{2}$
22,000	12	$7\frac{1}{4}$	6
35,000	18	12	10
45,000	24	16	14
70,000	36	24	21
90,000	48	32	27
110,000	60	38	33

Location of Lightning Arresters :—The aluminium arrester is now generally used in all high-voltage stations. Both the arrester tanks and the associated horn gaps may be located within the building or the horn gaps may be placed outside and the tanks inside, or both may be placed outside, provided that there is no danger of electrolytic freezing. Standard equipment of 27,000 volts and below are usually designed as complete units to be utilised inside the stations, whereas for those above 27,000 volts the horn gaps should preferably be installed outside the station and the tanks inside. Exception to this rule can be made where there is sufficient space in the station over the gaps.

The arrester tanks should naturally be located close to the line entrances. The horn gaps, when installed out of doors, may be placed on the roof of the building, if wall entrance bushings are used. The location of the arresters should also be such that the path for the discharge from the line conductors to the arresters and ground will be as straight as possible.

Clearance over Horn Gaps :—Wherever horn gaps are mounted inside the building, sufficient clearance should be allowed over them. There is no appreciable arc at the gaps, but, in abnormal cases, where the film has been allowed to get out of

order, the arc may be of considerable size. Where there are no bushes or inflammable apparatus, the following are the minimum clearances from the tops of horns to be allowed.

Sufficient room must be allowed between each pair of generators for completely dismantling one set. Sometimes for economy of space, however, the repairing area is kept at one end of the plant in close proximity to the workshop.

The Entrance Bay :—This should be at one end of the power-house. It should be ample in area to provide space in which apparatus may be placed when unloaded and when being repaired or assembling the parts of the machinery. The power-house should be readily approachable by the best available means of transporting the many equipment. With larger units the question of unit spacing and depth of excavation are important factors in determining the exact shape of draft tube and water passages. A long straight draft tube with a small angle of diffusion is the most efficient design.

Total Space :—"A reasonable average will be to allow for each machine room 13 c. ft. per K.V.A. of generator capacity. In a very compact lay-out this may go as low as 8 or 9 c. ft., while certain plants *on very liberal lines have occupied 25 c. ft. per K. V. A.*" (*p. 230, Gibson*).

In hydro-electric stations having high-tension transmission lines the space occupied by *switchgear and transformer* is on an average about 10 c. ft. per K. V. A. of generator capacity irrespective of transmission voltage, the limits being about 15 to 5 c. ft. per kW. of generator loss.

Ventilation :—Ventilation of the power-house is intended to prevent over-heating and to supply cool air to the generators. Ventilation is very carefully considered in each installation and such considerations are further based upon (1) climatic condition, (2) the type of structure, and (3) the details of construction of the generators.

In small and medium-sized plants, if the climate is cool, no special provision is necessary for ventilating the generators. The fans on rotors can circulate sufficient air drawn from the rooms to keep the machine cool. A loss of 1 kW. will heat up 1,800° c. ft. of air through 1° C when starting from ordinary room temperatures. When forced ventilation is not resorted to, the difference of temperature between outgoing and incoming cooling air should not exceed 18° C and is preferably kept lower, so that the actual air volume, necessary per kW. of generator loss, will be from 100-150 c. ft. per minute.

When the ventilating air for the machines is taken from the generator room, the maximum difference of temperature should not exceed 20° F (11·1° C) during hot weather. With forced ventilating schemes the difference of temperature should not be more than 30° F to 40° F (16·7°C to 22·7°C) between the incoming and outgoing air.

The generators must be kept clear of the structures which would interfere with proper connection. Such machines must not be kept in unventilated pit, which will act as a pocket for heated air and prevent adequate cooling of the bottom coils.

The outside air comes into the pit beneath the generator, which is sealed off in such a way that all air must be drawn through the machine frame, and thence discharged.

Air filters are not generally necessary with the atmospheric condition prevailing around a hydro-electric plant.

The air ducts in each machine should be kept as short and straight as possible, and unavoidable bends made with a wide sweep. The inside radius at the bend should not be less than three quarters of the duct-width. The duct-area should be such as to ensure a maximum velocity of 1,000 to 1,250 feet per minute. The difference in level between intake and discharge can generally be arranged to create an ample natural draught.

Large units specially in warm climate require large ducts through which outside cooling air may be admitted to the generators. The movement of the air to the generators is caused by fan vanes placed on the rotor; if this is not sufficient, motor-operated fans may be used in the air ducts. Each machine must be taken as an independent unit.

Dampers may be installed in the ducts, for controlling the air and with large stations it may be desirable to sectionalise the ventilating system.

Foundation :—On account of the vibration caused by the apparatus, it is best to use one-half of the allowed bearing pressures usually allowed for static load.

Careful sounding should be made to ascertain the character of the underlying strata. Separate foundations should be provided for different units so as to isolate any failure as far as possible. For soft alluvial soil, piling of wood or better concrete, either plain or reinforced, is almost always required.

Concrete is always used for foundation. A mixture of one part of cement, three parts sand and six parts gravel or broken *stone* has been used with perfect satisfaction for machine foundations.

Erection :—On small machines the foundation bolts and plates may be placed in position before the concrete is put in. They should be hung in place by a wooden template, and the bolts surrounded by stove pipe, conveyer pipe, scrap-iron pipe, several inches larger than the bolts themselves. This allows for mistake in location and variation in the machine parts, the holes being filled with the base are grouted. With large machines, however, it is better to have pockets in the concrete, large enough for the foundation plates to be dropped in. When these holes are filled in, and the base is grouted, they serve the further purpose of making a good bond between the foundation proper and the grout in and under the base.

The direct development utilises all the available fall at the dam, and the power station is located at its end or in the interior of the spill-way. This plan is recommended because of the concentration of the entire plant at one point and the consequential saving in the operation cost, and because of its securing the highest obtainable hydraulic efficiency of the power components, fall and flow ; by any other programme losses of both of these are incurred. Any diversion sacrifices a portion of the available fall by the slope in canals and flumes or the friction-head in pipe lines, while losses of flow are represented by leakage, evaporation, and ice conditions. When the water is passed at once from the upper pool through the turbines, no such losses occur. The conditions which determine this choice are the cost of the dam and embankments as compared with that of a lower dam and of diversion works ; also the extent and cost of flowage pond area ; the flood-flow conditions as affecting power-house ; the rise in the lower pool and the fluctuations in the working head.

Thus, while the direct development plan realises the highest percentage of flow and fall and represents the greatest simplicity of works and lowest operating charges, and, therefore, as a rule, the most economical, the conditions may sometimes be such that its adoption is prohibited by the first cost or by considerations of safety and of continuity of operations.

The Substructure :—The substructure of the power-house elevates the station above the lower pool, supports it, and forms the pits into which the water from the turbines is discharged and by which it passes into the tail-race or directly into the lower pool; power-house substructures are of practically one type, rectangular in plan, the length depending upon the number of power units, from 12 to 18 feet for each, according to the size of turbines, and the width conforming to that of the superstructure. The walls may be of masonry, monolithic or block

concrete the floor and roof of concrete on the downstream ends, are arranged for the reception of gates or stop logs to enable the emptying of any of the pits. The height of the substructure is regulated by the water-level in the lower pool; but the depth of water in the pits at the lowest level should not be less than five feet, as there should be a four-feet water cushion below the discharge ends of the draft tubes. There is no objection that the highest lower pool water should stand at the underside of the pit roof.

The **superstructure houses** the power equipment, and, depending upon the head, may be arranged for drowned or dry turbine installation. In the first case the turbines are placed in isolated bays into which the water enters freely with the upper level elevation, and passes through the turbines into the pit below; the pit roof forms the turbine-bay floor. The width and length of the turbine bays are regulated by the installation space required for the turbine, generally from 10 to 18 feet wide and from 15 to 40 feet long; their height is controlled by the upper level. The side walls or partitions are best constructed of concrete steel, and must be designed to resist lateral hydrostatic pressure existing when the adjacent bay is unwatered. The downstream end of the turbine bay is closed by a masonry or concrete wall or a steel-plate bulkhead of semicircular design with a radius equal to half the width of the bay and secured to the floor and partitions; either of these must resist the hydrostatic pressure due to the upper level head. The partition tops are connected by steel members, concrete steel beams or arches.

Essentials of Design in Building:—(1) Faithful adherence to the programme and its attendant requirements. (2) Faithful expression of the programme. (3) Stability, both real and apparent. (4) Beauty, resulting not from astonishment at mere size or ingenuity, but from the happy infusion of interest and variety into the elements of composition, always unified by harmony and proportion into a single idea.

Principles of Design:—It results from the faithful adherence to the programme to the last detail. By the programme reference is not only made to the material requirements of the client, but also to the mental appeal of the building.

Proportion:—The relative importance of the programme must be realised

By the true interpretation of relative importance, character is created. There are many shades of character in architecture; refinement, grandeur, gaiety, solemnity, vigour, restfulness, are always expressive of the purpose of the edifice.

Proportions will be determined by a variety of considerations; the relative importance of the various requirements of the programme; the traditional forms which have been evolved from the constructional development and use of materials; the traditional proportions which are associated with various historical styles; and the artistic taste of the designer.

Construction:—It must govern design. For, if a building expresses its construction truthfully, it will surely have the appearance of stability and repose, which are two of the great essentials in design.

Unity:—Beauty in architecture depends largely upon *unity* of form.

Symmetry:—An expression of unity will result from a regular or geometrical disposition of the elements on either side of a centre line.

Symmetry is highly desirable, but without destroying their proper functional sequence. In such cases, an *assymmetrical* composition for proper functional sequence may be adopted.

Harmony:—Finally, the expression of unity must be maintained by a consistency of stylistic treatment throughout the composition, with a harmony of proportion and scale in all features.

Ornament:—The consideration of ornament is far-reaching. It is always reasonable, however, to use features which have decorative value, so long as they are properly placed, and serve some definite purpose in the composition.

Truth:—In architecture there must be "material" and "moral" needs. The building must satisfy the practical requirements of the programme and it must be beautiful.

Materials:—The power plant buildings in all important stations are generally constructed with a concrete substructure and a structural steel framework enclosed with full brick walls as a superstructure.

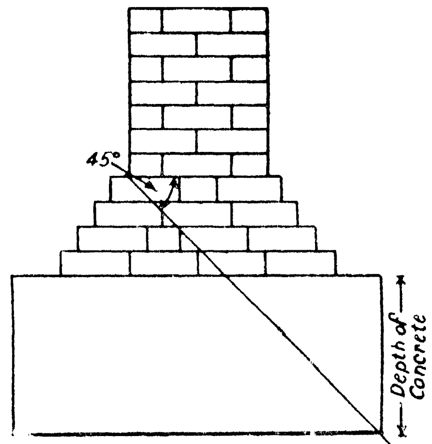
Detailed Consideration of Foundation

The functions of the foundation are: (1) to prevent the passage of water below the structure, (2) to afford rigidity of position to the superstructure. The design of foundation depends upon (*a*) the character of the material at its site as to hardness, strength and porosity, the height and weight of the superstructure, the maximum height of water to be ponded, and the effect of its overflow.

In good dry soil, which is compressible, but reasonably firm, an artificial foundation is necessary to increase the bearing surface. The standard type of foundation usual in these circumstances is obtained by extending the base of the wall to twice its thickness by projecting courses of brickwork called *footings*. Each course has offsets of 2½ ins. on each side, and below all is a bed of concrete of a depth calculated according to the nature of the soil, and exceeding the width of the bottom course of footing by 1 ft. 6 ins. on each side.

A method of obtaining the depth of concrete for ordinary foundations is shown here (Fig. 3). A line inclined at 45° is drawn from the junction of the wall and footings, and the point at which it cuts the perpendicular line of the concrete bed (which projects 6 ins. beyond footings) determines the required depth.

Fig. 3.



Method of obtaining the depth of concrete.

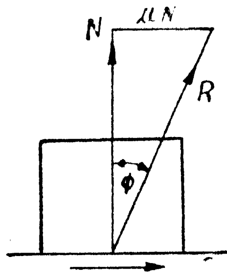
Testing Resistance of Earth:—The resistance of all soils should be tested. A good practical test can be made by the following procedure.

Construct a platform 3 ft. × 3 ft., and obtain four blocks 6 ins. × 6 ins. Place these on the ground to be tested; and under the four corners of the platform, on each side of and level with its top, drive in stakes. Now evenly load the platform, and note the effect of calculated loading from time to time; any settlement can be measured by comparison with the stationary stakes. As the area of the four blocks is equal to 1 sq. ft., the total load registers the resistance of 1 sq. ft. of foundation bed. From these data the necessary foundation for the structure may be determined. Soft or water-logged soils require special treatment.

Angle of Repose:—When a body slides over any other body and there is a normal reaction N between them (Fig. 4), the tangential force F opposing the motion is $F = \mu N$, μ being the coefficient of friction, and the resultant reaction between the bodies is R, which makes an angle ϕ with the normal force N, so that $\tan \phi = \mu$.

If a mass of dry sand is thrown into a heap, it is found that the sides of the heap take up a fairly definite slope; and if further sand is thrown on to the heap, the particles roll or slide down the slope. The angle of the slope to the horizontal is called the angle of repose, and tan $\phi = \mu$, the coefficient of friction of sand on sand.

Fig. 4.



If the sand is wetted with water, it is found that this slope may be increased very considerably until a certain quantity of water is added, when it will diminish rapidly. Twenty-five per cent. of water by weight added to sand makes it so slopy as to diminish the angle of repose to nearly zero. If the sand is mixed with clay, it is possible to obtain a greater angle of repose than with the dry sand. With care a trench can be cut into gravel or even into consolidated sand so that the face is almost vertical; but, if, in either of these cases, a disturbance is, let us say, accidentally, brought about, the side of the trench may slip in until the sand or gravel has taken up some slope which may be called the angle of repose for the particular condition. A bank may be made of earth or sand, which, when made, may assume an angle of repose of 45°, but after exposure the angle may diminish to 30°. For a substance like clay the angle of repose, when reasonably dry, may be 90°, that is, a trench may be cut with a vertical face; but, on the other hand, when clay rests on a hill-side and there is a possibility of the clay becoming softened by water, slipping may take place along planes inclined at a comparatively small angle to the horizontal plane. The reader will appreciate from his own observations and from a few simple experiments how difficult it is to give a definite value to the angle of repose.

TABLE I.

Values of the Angle of Repose.

Materials.	Angle of Repose (Degrees).
Very fine dry sand passing 1/50" mesh ..	30
Dry sand (Leighton Buzzard Standard) ...	22 to 24
Sand (crushed stone) dry, all passing a $\frac{1}{4}$ in. mesh...	33 to 35
Dry sand with 10 per cent. of water	36 to 80
Dry sand with 20 per cent. of water	85 to 90
Dry sand with 25 per cent. of water	0 to 10

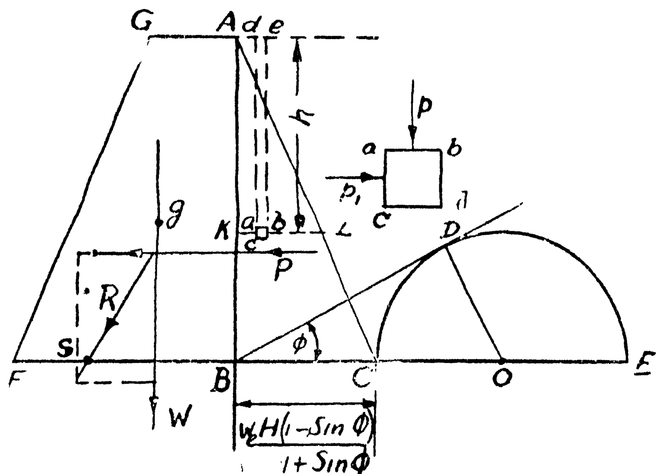
Materials.	Angle of Repose (Degrees).
Earth on earth	14 to 45
Earth on earth, dry sand, clay and mixed earth ...	21 to 37
Earth on earth, damp clay	45
Earth on earth, wet clay	15 to 20
Earth on earth, shingle and gravel	39 to 48
Masonry on moist clay	18
Masonry on dry clay	27
Masonry and brickwork with damp mortar ...	36
Dry masonry and brickwork	31 to 36

Friction of Any Plane in a Mass of Earth :—The fact that there is a possibility of dry sand taking up a definite angle of repose shows that there is friction between the particles resisting motion. In the theories of earth pressure, it is assumed that sliding cannot take place upon any plane until the tangential force tending to move any mass parallel to the plane is equal to, or greater than, the normal pressure on the plane multiplied by the coefficient of friction μ, which is equal to tan ϕ, or in other words, that the resultant force on any plane must make with the normal to the plane an angle at least equal to ϕ.

Rankine's Theory of Earth Pressures:—Pressure on a Retaining Wall :—Making the assumption that slipping in a mass of earth cannot take place on any plane until the resultant pressure makes an angle, with the normal, equal to the angle of repose ϕ, and assuming this to be constant for all planes, Rankine showed that if on any plane ab in a mass of earth having an angle of repose ϕ, Fig. 5, there is a vertical pressure p per unit area, then the horizontal pressure p_1 on the vertical plane ac is—

$$p_1 = \frac{p\,(1-\sin\phi)}{1+\sin\phi}$$

Fig 5.



Experiments carried out by a number of workers show that for

dry sand the results given by the experiments agree moderately well with the theory.

Let earth, having an angle of repose ϕ, be assumed level with the top of a wall having a vertical face AB. At any depth h, consider a small element abc. Above ab there is a column of earth of depth h, which may be assumed to rest on ab. On this assumption, the pressure on ab per unit area will be—

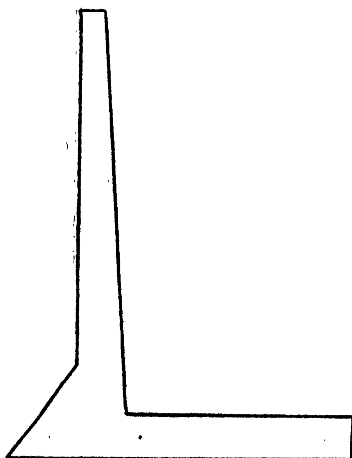
$$p = W_e h$$

W_e being the weight of one unit volume of earth. If W_e is taken as the weight per cubic foot and h in feet, then p is in pounds per square foot. The horizontal pressure per unit area on ac, that is, on the vertical face of the wall at the depth h, is

$$p_1 = \frac{W_e h (1-\sin \phi)}{1+\sin \phi}$$

The assumption that the pressure on ab is equal to the weight of earth above ab neglects any friction on the sides of the column abdc, and the assumption would appear to be on the safe side. If a tube be carefully filled with sand, the pressure on the lower end may be much less than the weight of the sand, but if the sand be disturbed, it may exert practically the full pressure on the lower end. Another illustration: If a box, with a hole in the bottom, covered by a flap, is filled with sand, the flap can be removed without much risk of the sand falling out; but if the box be struck and flow commences, the whole of the sand may quickly run out. If the sand is slightly damp, this arching effect is very considerably increased.

Fig. 6.



The worst case is to assume that the whole of the weight of the column abdc acts on ab, and it will generally be safe to assume this in designing retaining walls.

To diminish the pressure on the foundation of a cantilever wall, a toe is frequently projected as shown in Fig. 6

Retaining Wall, Supporting Earth Inclined to the Horizontal:—
When the earth surface is inclined to the horizontal, Fig. 7, at an angle θ to the horizontal, Rankine shows that, following the same assumptions as for the earth surface horizontal, the pressure p per unit area in a vertical direction, on the surface ab, Fig. 8, parallel to the earth surface and at a depth h feet, is $W_e\, h \cos\theta$; also, what he calls the conjugate pressure p_1 on the vertical face ac is parallel to ab and

Fig 7.

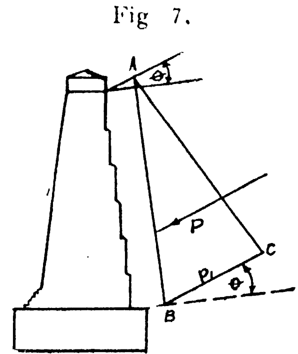
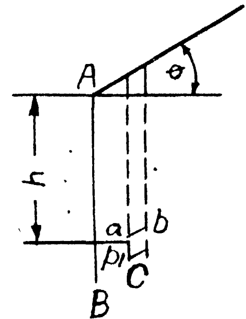


Fig. 8.



$$p_1 = W_e\, h \cos\theta \frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}$$

when $\theta = \phi$

$p_1 = p = W_e\, h \cos\phi$,

when $\theta = 0$

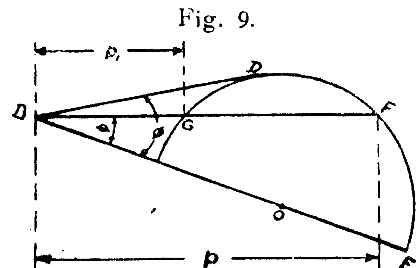
$$p_1 = W_e\, h \frac{1 - \sin\phi}{1 + \sin\phi}$$

The value of p_1 for any values of θ and ϕ can be found by the construction shown in Fig. 9. Draw any line BE and lines BD and BF making angles with it equal to ϕ and θ, respectively. Draw from any centre O, a circle touching BD in D.

$$\text{Then, } \frac{p_1}{p} = \frac{BG}{BF}$$

If, therefore, p is known, p_1 is easily found.

Fig. 9.



Now let BC, Fig. 9, be made equal to p_1. Then ABC is the total pressure per unit length of the wall. The resultant pressure p acts through the centre of gravity of ABC, and is parallel to the earth surface. The value of p can easily be found from the diagram, or $p = \frac{1}{2}\, AB\, p_1$.

$$= \frac{1}{2} W_e\, h^2 \cos\theta \frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}$$

which, when $\theta = \phi$, $= \frac{1}{2} W_e h^2 \cos \phi$

and, when $\theta = 0$, $= \frac{1}{2} W_e h^2 \frac{1 - \sin \phi}{1 + \sin \phi}$

Stability of a Retaining Wall:—Consider Fig. 5. Let g be the centre of gravity of the section of the wall. Then above the base, FB, the two forces acting are the horizontal force p and the weight W of the wall Combining these two, the resultant thrust R cuts the base BF at the point S.

This resultant R can now be resolved into a normal component N and a tangential component T. The normal component N causes direct thrust and bending upon the base, and T causes a shearing force which tends to make the wall slide. The stresses due to N and T can be found by the methods already discussed.

For stability against overturning, the resultant need only cut the base somewhere in the base FB, but it is desirable that it should cut it within the middle half, and if there is to be no tension on the base, within the middle third of the base.

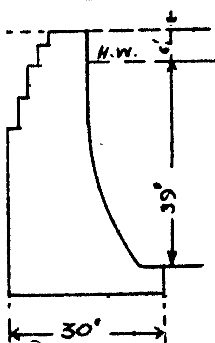
The Sliding of Retaining Walls and Dams:—The tangential component T of the resultant force R acting on any section of a masonry structure, it has already been seen, must be balanced by a tangential resistance equal to, or greater than, this force, or sliding will take place. The maximum resistance is μ N, where μ is the coefficient of friction. In the case of a retaining wall or masonry dam, T is the horizontal component of the earth or water pressure above the given section. For such walls the section on which sliding is most likely to take place is at the foundation, and cases of such sliding are by no means unknown. Walls resting on clay, which may become wet, or gravel, which may become impregnated with water under a hydrostatic head, may be liable to slip.

If a total hydrostatic pressure, N_1, either due to fissures or imperviousness of the foundation strata, can be exerted on the base of the wall, then the resultant vertical force acting on the base is $N - N_1$, and for no sliding

$$T < \mu (N - N_1)$$

When a clay foundation becomes wetted or a gravel foundation is impregnated with water, the coefficient μ may become small and sliding may take place. In the neighbourhood of rivers and with dock walls on bad foundations, the danger of sliding is a real one. Fig. 10 shows a concrete dock bottom having 51 ft. total height

Fig. 10



with 6 ft. below the dock bottom and 30 ft. wide at the base, which moved under lateral pressure a distance of 23 ft. when sandy clay, similar to the foundation, was filled in behind the wall. During the sliding the wall did not overturn. It was taken down and rebuilt of the same section, except that the foundations were taken 15 ft. below the dock bottom and no sliding took place. The student will readily satisfy himself that the wall was quite stable as far as overturning was concerned. If no water got behind the wall, the pressure due to earth could not have been more than about 0·4 of the weight of the wall, and the coefficient of friction was thus probably less than 0·4.

The danger of sliding can be minimised by excluding water, if that is possible, and by making the wall of extra thickness. Driving poles is of doubtful utility for this purpose. On rock foundations a smooth surface for the foundation should be avoided.

Fig. 11 shows a retaining wall at Sundarijal Hydro-Electric Station.

Depth of Foundations:—When foundations are built on sand, gravel, or similar material, an estimate of the safe depth below the surface of the bottom of the foundation can be made. Let p be the pressure per unit area on the toe of the wall, Fig. 12. Then the pressure p will induce a lateral pressure p_2

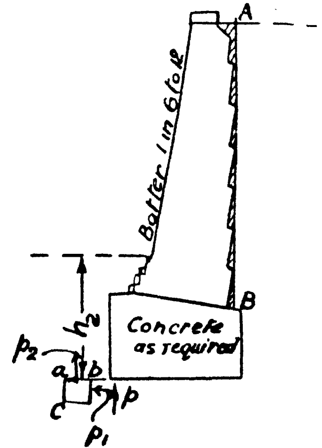
$$p_1 = p\,\frac{1-\sin\phi}{1+\sin\phi}$$

The lateral pressure p_1 will induce a vertical pressure $p_2 = p_1\,\dfrac{1-\sin\phi}{1+\sin\phi}$, which must be balanced by the weight of earth above ab.

$$\text{Then } W_e h_2 = p_2 = p\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$$\text{or, } h_2 = \frac{p}{W_e}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

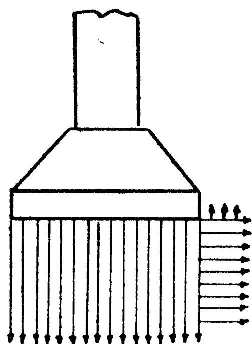
Fig. 12.



It is often difficult to foresee the nature of foundations, particularly for retaining and dock walls, and where there is uncertainty it is desirable to specify on drawings "foundations as required."

Depth of Footings :—If a vertical pressure of p lbs. per sq. ft. is applied to a horizontal surface by the base of a column, such as that is illustrated in Fig. 13, according to Rankine's theory of earth pressure, there will be a horizontal pressure in the soil equalling

Fig. 13.



$$p \times \frac{(1-\sin\phi)}{(1+\sin\phi)},$$

where ϕ is the angle of repose or natural slope of the soil. This expression may also be written as p. $\tan^2 (45^\circ - \phi/2)$, a form more useful for slide rule calculation.

From this horizontal pressure there will result an upward pressure equalling p. $\tan^4 (45^\circ - \phi/2)$, and unless the soil by the side of the column base is sufficiently loaded, it will be pushed up as the base sinks down. If, for example, the natural slope is 35°, $\tan (45^\circ - 17\frac{1}{2}^\circ) = {\cdot}5206$, $\tan^2 \phi = 0{\cdot}27$. If the downward base pressure is 2 tons = 4,480 lbs. per sq. ft., the corresponding horizontal pressure is 1,210 lbs. per sq. ft., and the resulting upward pressure is 327 lbs. per sq. ft.

If the soil weighs 100 lbs. per cu. ft., it would thus require a depth of 3 ft. 3 ins. to ensure that the soil surrounding the base will not rise up as the base sinks.

Physical Characteristics of Rocks

Rock.	Safe load on bearing surface per square ft.	Weight per cu. ft.	Crushing strength per sq. ft.
Granite ...	15 tons	180 lbs.	750 tons.
Sienite ...	...	,,	,,
Gneiss ...	...	,,	700 tons.
Sienite gneiss ...	...	,,	,,
Trap ...	...	,,	,,
Basalt ..	...	,,	,,
Greenstone ...	...	,,	,,
Sandstone ...	...	150 lbs.	600 tons.
Marble ...	...	168 lbs.	500 tons.
Limestone (hard) ...	9 tons	168 lbs.	500 tons.
Slate ...	...	175 lbs.	600 tons.

Physical Characteristics of Alluvials

Material.	Weight per cu. ft.	Bearing per sq. ft.	Angle of repose.	Coefficient of friction.
Gravel ...	90-106 lbs	2-3 tons	38°	0·78
Sand, dry & loose ...	90-106 lbs	,,	28°	0·53
Sand, wet ...	118-129 lbs.	,,	,,	,,
Clay, dry ...	119 lbs.	4-6 tons	..	...
Clay, in lumps ...	63 lbs.	,,	45°	...
Clay, damp ...	...	,,	,,	1·0
Clay, wet ...	...	,,	15°	0·27
Loam, dry and loose	72-80 lbs.	,,	35°	0·70
Loam, wet ...	66-68 lbs.	,,	,,	,,
Mud ...	101-110 lbs.	,,	Zero	,,
Gravel and loam ...	...	2-3 tons	38°	0·78
Gravel and sand, dry.	...	8 10 tons	45°	1·0
Loam on moist clay	..	...	,,	,,
Loam on wet clay ...	...	...	17°	0·3
Clay on gravel ...	...	...	45°	1·0
Peat ...	...	...	20°	0·36

In rock its hardness, stratification, condition and shape of surface determine the foundation.

Footings, however, should be placed a sufficient distance below the surface to be beyond the effect of the weather. This distance will vary with both soil and climate. If clay is so situated that in the summer it will dry, shrink, and crack for a distance of 6 ft. below the surface, the footings must be placed deeper than 6 ft. If frost does not penetrate more than 4 ft. into gravel, such a depth would be suitable for footings therein.

Load Which Foundation Will Bear

Safe Load on Ordinary Foundations

		Tons per sq. ft.
1.	Rock, moderately hard, strong as the strongest red brick	9·0
2.	Rock of the strength of good concrete ...	3·0
3.	Rock, very soft	1·8
4.	Moist clay and sand	1·36
5.	Coarse sand and dry clay	2·25

		Tons per sq. ft.
6.	Firm stone on dry clay	3·18
7.	Loose beds with piling	1·82
8.	Loose beds with concrete	2·75
9.	Brick, stock, mortar	2·5
10.	Brick, stock, Portland cement, 1 to 1 ...	8·0
11.	Concrete, Portland cement, 1 to 6 ...	15
12.	Rubble on lias	4

Intensity of pressure on a rock foundation should, at no point, exceed one-eighth pressure which would crush the rock.

Buildings seldom press with a weight of more than 1 ton per square foot on foundations.

Safe Load on Materials

		Cwts. per sq. in.	Tons pe sq. ft.
1.	Portland cement, concrete 5 to 1 ...	2·0	15
2.	Portland cement, concrete 10 to 1 ...	1·0	7·5
3.	Mortar, ordinary	·5	3·5
4.	Brick in mortar	·5	3·5
5.	Brick in cement	·8	5·75
6.	Granite	10·0	72
7.	Limestone	9·0	65
8.	Sandstone	5·0	38
9.	Rubble	·5	3·5

Safe Load on Bearing Surface for End of Girders

	Tons per sq. ft.
Brick, stock, in cement	6
Brick, stock, in lias	5
Brick, stock, in lime	4
Earth, compact	2
Earth, made	1

Safe Load on Stone Walls and Columns

	Ashlar Walls, single bedstones, columns, diameter = $\frac{1}{8}$ height. Lbs. pe2 sq. in.	Block in course columns, diameter = $\frac{1}{12}$ height. Lbs. per sq. in.
Granite ...	712 ...	570
Hard stone ...	356 ...	280
Medium ...	214 ...	142

Safe Load on Brick Walls and Columns

	Walls not less than 18 ins. columns, diameter-$\frac{1}{6}$ ht	Walls under 18 ins. columns, diameter-$\frac{1}{8}$ ht.	Columns, diameter $=\frac{1}{8}$ to $\frac{1}{12}$ ht.
	Lbs. per sq. in.	Lbs. per sq. in.	Lbs. per sq. in.
Brick in motar	72	36	—
Brick in cement	108	72	—
Brick in Portland cement	142	108	44
Rubble mortar	58	—	—
Rubble cement	72	—	—

Safe Loads on Floors

	Lbs. per sq. ft.
Dwelling-rooms	51
Stairs	82
Work-rooms	82
Warehouses	112
Library	72

Foundation on Different Soils

In any structure begun at different levels, larger blocks should be used for the deeper parts, if the material admits of it, so as to reduce the number of mortar joints and the risks of unequal settlement.

With sand or gravel foundation overlying clay on a slope, intercept water by drain on upper side of building.

Least depth to escape effects of heat and frost, from 3′ to 6′ according to climate.

Rock:—Good foundation, must be level; should be levelled, if necessary, in steps. The uneven parts should be filled up with large stones, firmly built with strong cement, or with concrete.

Gravel:—The best foundation.

Sand:—Good foundation, if dry and not liable to be washed away, but this easily occurs; drains with leaky joints may cause a subsidence, or any disturbance of the water-level in the stratum, whether by natural or artificial means, such as pumping operations connected with deep foundations, even at a great distance.

Clay:—Generally very treacherous and damp; the foundation must be deep.

Hard overlying soft ground:—If care is taken that the pressure per unit of area is not greater than the firm layer will bear, it may be wiser to build on it, sinking into it as little as possible.

Soft ground overlying hard ground:—If the stratum of soft ground does not exceed 15′ to 20′, it will generally be cheaper to sink down to the firm ground; if not more than about 30′, drive piles or sink wells of masonry. If of indefinite depth, the platform must be supported by friction against the sides, and be, therefore, of considerable thickness.

Made ground should never be trusted for the support of much weight, even though it may have lain undisturbed for years.

Sometimes foundations of buildings are on black cotton soil—*e.g.*, a part of the Sundarijal Power-House at Nepal is on black cotton soil. Its treacherous character is well-known.

Discussion on Foundations in Black Cotton Soil:—"Mr. Ishwari Prasad, an Executive Engineer in the C. P., suggested a novel design of foundation, which was successfully adopted in the case of some minor buildings. It consisted of providing a block foundation of shallow depth, say about 2′, filled with rammed gitti and morum, and then constructing the building on this base. He remembered a building of this type, *viz.*, the out-houses of the old Commissioner's Bungalow at Hoshangabad, which did not develop any crack. Mr. Ishwari Prasad got his idea from observing that in roads passing over black soil country, although there were wide cracks across the soil, they never showed themselves over the metalled surface.

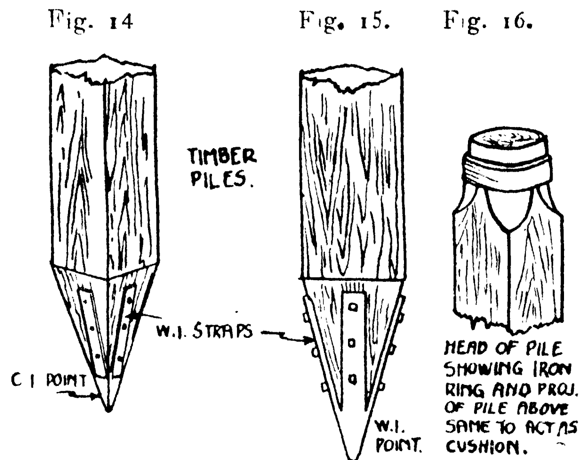
The method of founding generally adopted on the B. N. Railway in black cotton soil was to excavate the whole area to be covered by the building to a depth of 7 feet (sometimes in very bad soil a little deeper), and to fill in the whole area with suitable material which would not expand or contract much under varying conditions of moisture and heat such as morum watered and rammed, sand or ashes, particular attention being paid to the consolidation of the filling material under the position of the main and the cross-walls of the structure. By this means the disruptive forces within the structure were eliminated. In very bad cases where cracks of any magnitude were liable to occur below the greatest excavated depth, the excavation was usually paved with laterite blocks, boulders or brickbats to prevent the filling from escaping down the cracks. After completing the filling up to ground level, shallow trenches some two feet deep having been kept for the foundation of walls, shallow foundations

were put in the ordinary way." (Mr. Ishwari Prasad,—p. 45 ; F. S. Hughes in the Journal of the Institution of Engineers, India—pp. 48-49, and Vol. X—May 1931.)

The foundation should weigh $2\frac{1}{2}$ to 4 times that of its engine, depending whether horizontal and vertical type, also on the outside forces, belt pull, direction, etc.

Pile Foundations

Bearing piles of timber vary from 9 in. to 18 in. square, depending on their length, which, as a common rule, should not exceed 20 times the least dimension. They should be of whole timbers, straight grained, with bark or rough projections removed, free from large knots, and used with butt and downwards. The bottom ends must be pointed, preferably with wrought-iron shoes as Fig. 14 or as in Fig. 15, which has the point of cast iron and the straps of wrought-iron to prevent the points being broken or crushed when being driven against hard substances.



The point of the shoe must be axial to the pile, otherwise it will be apt to deviate from the vertical during driving.

The head of the pile should have a wrought-iron hoop or ring, placed not more than $1\frac{1}{2}$ ins. or less than $\frac{3}{4}$ in. from top, as shown by Fig. 16, to prevent it from splitting during driving.

The theoretical *bearing capacity* of timber piles equals cube root of fall of hammer in feet × weight of hammer in pounds × constant 0·023, divided by constant 1 + sinking distance per stroke in inches, which for "driven to refugal" conditions are $\sqrt[3]{20} \times 2{,}000 \times 0{\cdot}033 \div 1{\cdot}5 = 64$ tons. The actual loading should not exceed one-half of theoretical when the pile stands in gravel, clay, and sand ; one-fourth of theoretical when the pile stands in clay and sand ; one-tenth of theoretical when the pile stands in mud. To drive bearing piles in compacted sand is difficult and generally requires clearing by water jet.

The number and disposition of piles depend on the loads to be supported, their position on site, and the resistance of the driven piles, all of which have to be carefully designed and calculated.

Bearing piles must be driven until the shoes are embedded in a hard stratum, or until the pressure or friction of the ground on their sides is sufficient to prevent further penetration under a load a little less than the failing load for which they are calculated. A sufficient resistance is considered by engineers to be obtained when the last blow does not sink the pile more than $\frac{1}{4}$ in. There is another test which appears to be sound—

W = weight of ram; H = fall of ram; and D = set of pile with last blow.

Then—

Either W = 8 cwt., H = 5 ft., D = $\frac{1}{5}$ in. in 30 blows.
Or, W = 15 cwt., H = 15 ft., D = $\frac{1}{4}$ in. in 10 blows.
Or, W = 8 cwt., H = 30 ft., D = $\frac{1}{5}$ in. in 10 blows.

It must be noted that D is the set of the last blow of the total number shown, and not the set caused by 10 or 30 blows.

There are several formulæ for estimating the load that a bearing pole will support, the simplest one that gives a very fair result is that of Major Saunders, U. S. A.

$$L = \frac{Wh}{8d}$$

where, L = safe load in cwts.;
W = weight of ram in cwts.;
h = height that ram falls in inches;
d = distance driven by last blow in inches.

This formula allows a factor of safety of 8. It will be advisable now to consider the ram of the driver, *i.e.*, the weight that in falling a certain distance drives the pile into the ground. It is considered in practice that a light ram with a long fall is best for clay soils, and a heavy ram with a short fall for sand and sandy soils, and a great number of light blows is preferable to a small number of heavy blows, especially in sand.

The weight of the ram varies from 5 cwts. to 40 cwts., and is generally 10 cwts. to 20 cwts. for ordinary work, but the weight is regulated by various rules and formulæ, as follows:—

(*a*) A rough rule is that the ram should be not less than or more than $1\frac{1}{2}$ times the weight of the pile.

(*b*) Diameter or least dimension of pile in inches *minus* 5 multiplied by 3 = weight of ram in cwts.

(c) Length of pile in feet multiplied by sectional area in inches divided by 4 = weight of ram in lbs.

Consolidating or short piles are usually from 6 ft. to 12 ft. long, more slender than bearing piles, and are driven into the ground to so compress and consolidate it as to make it capable of being built upon in conjunction with the piles. They are usually placed as close together as possible, but far enough apart to avoid the tendency of one pile, while it is being driven, causing the adjacent ones to rise. Usually, 2 ft. 6 ins. centres will be suitable.

Sheet piles are used to enclose area of foundations which are liable to escape laterally, and so cause subsidence, or in some cases to protect a site from water.

Walls

Isolated walls are designed primarily to resist earth or water-pressure. They are usually about one-fifth of their height in thickness, and in some cases battered, or they may consist of a thin wall with buttresses or piers at intervals.

Collective walls are primarily space enclosing elements which form rooms, or collections of rooms, in a building ; consequently, their form and dimensions must first be determined by the requirements of the plan.

The first consideration in the design of the wall itself, as part of a composition, is its thickness. This will be determined by the requirements of construction, climate, effect, and decoration.

Construction :—In any one building there may be walls of varying thicknesses. External walls will usually be thickest, since, besides protecting the inside of the building, they have to resist the oblique thrust of a roof and the eccentric loads of floors. Internally, it is necessary to distinguish between partitions and load-carrying walls. The latter will be required to resist actions which tend to crush them, such as loads from floors, and actions which tend to overturn them, such as oblique thrusts from vaults and arches. In a good plan, these oblique forces will be resisted by skilfully-arranged cross-walls of normal dimensions or by the balancing of one vault or arch against another.

Stability may be attained by the use of piers or buttresses at regular intervals. Once the general proportions of these are settled, their actual dimension, if in brick walls, must be a brick dimension, in order to avoid waste and unnecessary labour.

Walls and Piers

Purpose of Walls :—Walls are required to enclose a space, and for the division of a structure into a number of apartments ; or as supports to carry the weight of secondary structures, such as floors and their loads, partitions, and roofs.

Before walls are built, many conditions affecting their stability have to be considered : the materials of construction, their thickness, height, and length ; the nature of their loading and its distribution.

In considering their loads, the weight of the walls themselves, and the weight of interior parts that are transmitted to the walls, must be included ; also the thrust of arches, corbels, flights of stairs, etc.

Thickness of Walls :—The thickness of walls largely depends, apart from their loads, upon the relation of their height to their length, and to the spread of their bases to prevent overturning. An isolated wall, such as a boundary or parapet wall, would require a greater thickness, and consequently a wider spread of foundation, than the walls of a residence, which receive much support from intersecting, joining, and returned walls.

Thickness is also dependent upon the weather-resisting properties of the materials with which the walls are built. The walls should be sufficiently thick to prevent the too-rapid conduction of heat, both artificial and natural ; also to prevent the passage of sound from apartment to apartment.

The Thickness of External Walls :—It should not be less than one-sixteenth of the height of the story in the case of residences, and not less than one-fourteenth in the case of warehouses.

The following particulars are a good guide for the requisite thickness of walls under ordinary circumstances :—

Walls of two stories not exceeding a total height of 25 ft., one brick thick.

Three stories in height : top story, one brick thick ; lower stories, $1\frac{1}{2}$ bricks thick.

Forty feet to 50 ft. in height : top story, one brick thick ; lowest story, two bricks thick ; and the intermediate stories, $1\frac{1}{2}$ bricks thick.

Fifty feet to 60 ft. in height : two bottom stories, two bricks thick, and the upper stories $1\frac{1}{2}$ bricks thick.

Piers:—In forming window or door openings or recesses, a wall is divided into a number of piers, which becomes the subject of concentrated loads. The openings between the piers are spanned by girders, lintels, or arches, which carry the weight of the work above and transmit it to the piers; the various thrusts at the abutments of arches have also to be considered.

The sectional area of the piers, that is, their length multiplied by their breadth, must be sufficient to carry the concentrated loads, and it frequently becomes necessary to thicken the wall at these points.

Isolated piers receiving no support from abutting walls need to be of a greater sectional area than connected piers. Their height is relative to their sectional area, and varies with different materials to a point, where it gradually becomes weaker owing to its liability to buckle under its own weight, apart from its load. No isolated pier should be built higher than from ten to twelve times its least diameter.

A pier 10 ft. high will only carry half the weight that it will carry when only 1 ft. in height.

The density and hardness of the material with which a pier is constructed affect its stability.

Stock brickwork crushes under a load of about 30 tons per sq. ft., half this load will cause failure by cracking, which will eventually destroy the structure. A *safe load* would be one-fifth of that which causes fracture, and one-half of this result for a pier 10 ft. in height.

The *foundations* for brick piers in the ordinary way are the same as for walls, the spread of the footings being equal to twice the least thickness of the pier. This again depends upon the load it is called upon to support.

An easy rule for calculating the number of courses of footings for any thickness of wall, is to reckon one course of footings for every half-brick in the thickness of the wall, the bottom course of which will be twice the thickness of the wall; as an example, a one-brick wall has two half-bricks in its thickness; therefore, there will be two courses of footings, and the bottom course will be two bricks wide.

Again, a two-brick wall has four half-bricks in its thickness; there will be four courses of footings, the bottom course being four bricks wide.

Each course of footings is set back $2\frac{1}{4}$ ins. from the face of the course below, forming a series of steps on each side cf the wall. These steps are called *offsets*.

Floor :—No combustible material of any kind should, if possible, be used in the construction of a power-house. The floor should be of concrete. Ample room should be provided on the operating floor so that each machine can be readily dismantled, repaired or removed. A ½″ to 1½″ open joint all the way round machine foundations subjected to vibration, will, in a large measure, isolate the vibrations in a building walls. The joint may be filled with asphalte or similar material, but no harm is done by having it open.

Walls :—Walls may be of masonry, *e.g.*, of reinforced concrete or of brick with a steel skeleton framework. The floors and walls are properly designed not only to support safely the structure and equipment but also to be rigid against vibration.

Roof :—Roof of the building should be of the most substantial and fireproof character, of steel trusses, carried on the side of the walls or on the steel columns. The slope should not be excessive, 2 inches per foot being sufficient with gravel covering. Corrugated iron roofs are seldom used in power-houses in America.

Windows :—Good lighting is imperative, and large windows are, therefore, essential. They should be placed symmetrically with reference to the generating units, and should be such as to harmonise with the building, arched windows being very generally used. The window sashes should preferably be metallic, and the glass reinforced with wire-netting so as to prevent shattering when open. Ribbed or non-transparent glass is also desirable because it keeps out the intense rays of the sun.

Doors :—One of the openings should be of sufficient size to admit a railroad car, and tracks should also be provided. Very often these doors are of the rolling type, this design being most economical as regards space.

Travelling Crane :—This should always be made for supporting the track for a travelling crane which should span the generator room and run the full length of the station.

The track is generally supported on pilasters in the outside wall on steel columns separating the generator and the switch rooms. There should be ample head rooms allowed so that the various machine parts can be readily removed when repairs are to be made.

Miscellaneous Rooms :—Repair rooms, store rooms, office, toilets, etc., should be provided.

Size of Down Pipes :—A common rule for the size of down pipes is to provide 1 sq. in. of cross-section for each 100 sq. ft.

of roof area. This rule would indicate a 1 in. diameter pipe to be sufficient for 78½ sq. ft., but a somewhat larger pipe should certainly be put in for such an area. The rule would also give a 2 in. diameter pipe for about 300 sq. ft., and a 4 in. pipe for, say, 1,250 sq. ft.

Depth of Concrete in Foundation:—Allow 6″ for 10′ height of wall.

Brief Notes on Stair Design:—Avoid introducing winders into stairs, whenever possible.

When winders are unavoidable, they should be placed at the bottom rather than at the top of the flight, the narrower end being made as wide as possible. When spacing for winders, measure the width of tread 18 inches from the free-end, as this distance from the handrail forms the line of tread.

The inclination of a stair should be kept between 30° and 45° to the horizontal.

The product of the width of the tread and the height of the riser in inches should be 66 (approximately).

The tread should not be less than 2 inches.

The height of the top of the handrail, vertically measured over the face of the riser, should be in the neighbourhood of 2 ft. 7 ins.

Head room of at least 7 ft., vertically measured over the face of the riser, should be allowed.

Ample lighting should be arranged in every stairway, a skylight and counter-light being particularly suitable for this purpose.

It is undesirable to have more than fourteen steps in one flight.

A single step should never be introduced for any purpose.

No stair should be less than 3 ft. wide.

In extremely wide stairs a centre handrail should be provided.

Stone and Concrete Stairs:—The dog-legged type of stair, with a central brick dividing wall, lends itself best to construction in stone or concrete. When stone is used, a fairly fine-grained sand stone should be selected, and the steps may be, either, the solid rectangular or spandril step, with square end seating; both ends should be built, at least, 4½ ins. into the side-wall. The going should not be less than 9 ins., nor the rise more than 7 ins., and a metal handrail should follow the rake of the stairs from top to bottom.

Efflorescence on Brickwork :—The unsightly white incrustation on brickwork is found on analysis to consist of sodium carbonate, with sometimes a trace of sodium sulphate, and (rarely) calcium sulphate. There is no adequate remedy for the trouble known at present. It is suggested that the sulphur content of the soil may be one of the contributory factors. It is a peculiar fact that the efflorescence is never found on a loose stack of bricks, but only on finished walls in which cement has been employed. It has been suggested that a connection may exist between the salts used to accelerate or retard the rate of setting of cement, and the presence (and nature) of the incrustation.

Installation of Machinery

Installation :—The foundation and bolts, however, are nearly always provided by the power-station builder. Whenever the erection is done by the builder, the manufacturer's instructions should be very carefully followed. Small generators can be shipped completely assembled, but larger machines must be assembled more or less completely on the foundations. It is common practice with the larger sizes to install the form-wound stator coils, and even to stack the laminated cores, as the machine is assembled on its foundation.

Foundations for water-wheel generators are commonly made of concrete, and are usually an integral part of the power-house substructure. The foundation should season for two or three weeks before the machine base is installed. Bolts for anchoring the base should be cast into the concrete and accurately located by means of a template. It is good practice to provide each bolt with a large steel-plate washer to increase its holding strength, and to surround it with an iron pipe of sufficient size to give some lateral freedom. The foundation should be built up to within about 1 in. of the grade of the bottom of the machine base, to allow for grouting. After the base is set, the grouting should be given ample time to season thoroughly before the machine is placed upon the base. It is essential that the foundation extends under the entire base and positively prevents any deflection, since the manufacturer does not intend that the base be rigid enough to maintain proper alignment of the machine unless uniformly supported. The careful levelling and uniform grouting of the base are, perhaps, the most important details in the erection of the machine.

The best grout mixture is half sharp, clean sand and half cement. It should be thin enough to flow readily and should be well puddled into place. Before pouring, all dust and trash should be cleaned off and the foundation thoroughly wet down. It is better to use a fairly slow setting cement on large castings. In some cases cement for grouting has been set aside and aged a year before using. Fresh or quick-setting cement may heat enough, while setting, to cause expansion and distortion of large castings. A record of the grout and room temperature should be taken as a check.

CHAPTER XIV

HYDRO-ELECTRIC INSTALLATIONS OF INDIA

In chapter III, we have discussed the various types of hydro-electric lay-out. In this we give " illustrations of various falls utilised so far—at least all the important falls as follows :—

(1) Mahora and Ranbir Canal falls at Jammu and Kashmir. The former utilises the Jhelum River and the latter uses the fall from the Ranbir Canal.

(2) The Uhl River is utilised in the Shanon Power-house at Mundi.

(3) The River Cauvery is utilised by erecting a huge dam at Shiva Samudram.

(4) The three Tata Schemes at Bombay collect the monsoon water, in huge reservoirs, which is utilised throughout the year.

(5) The Gokak fall is utilised for the mills at Gokak.

(6) The great Mettur Dam is used at present for a comparatively small hydro-electric plant at Mettur.

(7) The fall at Pykara is utilised for power purposes.

(8) The fall at Darjeeling is utilised for light and power at Darjeeling.

(9) The fall at Mussoorie is similarly utilised.

(10) The fall at Simla is similarly utilised for supplying light and power at Simla.

(11) The Ganges Canal falls are used to supply power and light over a considerable part of U. P.

(12) The Naini Tal or lake is similarly utilised by the Municipality for supplying light and power.

(13) The two falls at Nepal—at Phurping and Sundarijal—are utilised for similar purposes.

Now, we shall proceed with the discussions of these falls in order.

1. Kashmir Hydro-Electric Schemes :—There are two hydro-electric schemes developed in Kashmir. Out of these, one is

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